6. Connection Qualification

6.1 Scope

This chapter provides performance qualification data for various types of connections, together with criteria for analysis and design of connections for the upgrade of existing steel moment-frame (WSMF) structures. Included herein are general criteria that are generic to most connection upgrade types, and recommendations for specific connection upgrade details of connections intended to be prequalified for use in seismic upgrades. Each of the connection prequalifications is limited to specific conditions for which they are applicable, including member size ranges, grades of material and other details of the connection. Also included in this chapter are procedures for qualification of connections and connection upgrades, which have not been prequalified or are proposed for use outside the limits of their prequalification as set forth herein.

Commentary: The 1988 Uniform Building Code (ICBO, 1988) introduced a single pre-qualified moment connection design, representative of prevailing west coast practice at the time. The "qualification" of this connection was based primarily on the research of Popov and Stephen in the early 1970's, and the belief that this connection was capable of providing acceptable strength and ductility for service in all frames that otherwise met the provisions of the building code. The UBC pre-qualified connection was subsequently adopted into the 1992 AISC Seismic Provisions and then into model codes nationwide. Although the building codes did not formally adopt the pre-qualification of this standard connection until the late 1980s and early 1990s, this connection detail had seen widespread use in WSMF construction since the 1970s.

The discovery of many fractures in buildings incorporating this standard detail, following the Northridge earthquake, demonstrated the ineffectiveness of the pre-qualified connection as it was being used in modern practice. Subsequent research conducted under this project, and by others, has demonstrated that many types of connections that have the strength to develop the plastic moment capacity of the connected elements, do not have the capability to do so in a ductile manner over repeated cycles of loading. Further, this research has shown that inelastic deformation demands in some frame structures can be significantly larger than those that have historically been presumed as the basis for the codes.

Following the 1994 Northridge earthquake, the pre-qualified connection contained in the building code was deleted by means of an emergency code change. In its place, a provision was substituted requiring that the designer demonstrate that whatever connection was used is capable of sustaining the necessary inelastic deformation demands. Qualification of this capacity was by prototype testing. In the time since, a significant number of connection assemblies have been tested, allowing new prequalifications to be developed.

Those prequalifications that are applicable to the upgrade of existing structures appear in this document.

Although a number of prequalified connection upgrades are available, it is conceivable that designers may wish to utilize other connection upgrade designs or to use a pre-qualified design under conditions that are outside those for which they have been prequalified. In these cases, a project-specific, qualification-by-test procedure is still required. The requirements for such a qualification procedure are also given in this chapter.

Finally, this chapter presents qualification and modeling data needed for the assessment of performance of the typical pre-Northridge style connection and of various types of simple gravity connections, for use in performance evaluation of existing structures.

6.2 Performance Data for Existing Connections

This section provides modeling criteria and performance data for use in assessing the performance of existing moment-resisting and simple connections typically found in existing welded steel moment-frame buildings. These connections are not prequalified for use in the lateral-force-resisting systems of new structures. For each connection type, the following quantities are defined:

q_{SD} = median total connection drift angle at which strength degradation occurs, radians. For existing brittle connections, this corresponds to the median estimate of drift angle at which brittle fracture initiates

q_{IO} = median drift angle capacity for Immediate Occupancy performance, radians

q_U = median drift angle at which connection looses gravity load carrying ability, used as the limit state for Collapse Prevention performance

f = a resistance factor applied to q_{IO}, or q_U, as appropriate

6.2.1 Welded Unreinforced Fully Restrained Connection

The data contained in this section applies to performance evaluation of existing buildings with the typical welded, unreinforced, moment-resisting connection, commonly present in WSMF buildings constructed prior to the 1994 Northridge earthquake. Figure 6-1 presents a detail for this connection. It is characterized by rolled wide flange beams connected to the strong axis of wide flange column sections, with the connection of the beam flanges to column flange through complete joint penetration (CJP) groove welds. Welding has typically been performed using the Flux Cored Arc Welding process and with weld filler metals without specific rated notch toughness. Weld backing and weld tabs are commonly left in place. Beam webs are connected to the column with a single plate shear tab, welded to the column and bolted to the beam web. In some forms of the connection, there are supplemental welds of the shear tab to the beam. Doubler plates, reinforcing the shear capacity of the column panel zone, and beam flange continuity plates at the top and bottom of the panel zone may or may not be present.

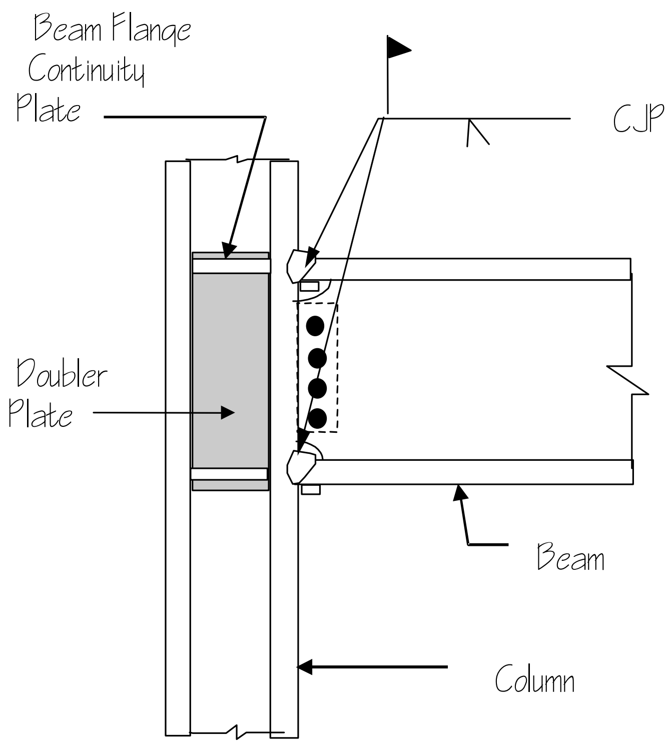


Figure 6-1 Welded Unreinforced Fully Restrained Connection (pre-1994)

Commentary: The data presented in this section is not specifically applicable to forms of this connection that employ weld metals with significant notch toughness. Some older buildings, particularly those erected prior to about 1964, may have welds deposited by the Shielded Metal Arc Welding (SMAW) process. Some such welds may have significant notch toughness, on the order of 40 ft-lbs at normal service temperatures. Limited testing of such connections indicates that they may have better inelastic deformation capacity than do connections employing weld material with lower notch toughness. Refer to Section 6.6.1 for data on connections with notch-tough weld metal.

The performance data provided in this section also is not specifically applicable to forms of the connection in which the beam web is directly welded to the column flange. Limited testing of such connections indicates that they are capable of providing somewhat better inelastic deformation capacity than similar connections with bolted beam webs. However, there are not sufficient data available to permit separate performance qualification of this connection type. The performance data provided herein may be conservatively applied to that connection type, or alternatively, project-specific qualification testing of such connections may be performed.

The connection performance data contained herein has been based on testing of connection assemblies in which the beams are connected to the major axis of the column. Connections in which beams are connected to the minor axis of columns are known to have similar, and perhaps, more severe vulnerability than major axis connections. However, insufficient data are available to permit quantification of this performance. Connections employing box columns are beyond the scope of this section.

6.2.1.1 Modeling Assumptions

6.2.1.1.1 Linear Analysis

Elastic analysis models of structures with Welded Unreinforced Fully Restrained Connections should be based on the assumption that the connection provides a fully rigid interconnection between the beam and column, located at the centerline of the column. Alternatively, realistic assumptions with regard to panel zone flexibility may be made, as indicated in Section 3.5.2.2.

6.2.1.1.2 Nonlinear Analysis

Nonlinear analysis models of structures with Welded Unreinforced Fully Restrained Connections should be based on the assumption that the connection provides a fully rigid interconnection between the beam and column, located at the centerline of the column, until the connection panel zone, the beam or the column yields, or a total interstory drift angle q_{SD}, from Table 6-1 is reached. The expected yield strength of the material, as indicated in Section 2.5 should be used to calculate the yield capacity of beams, columns, and panel zones. If yielding occurs at total interstory drift angles less than q_{SD}, the yielding element should be assumed to exhibit plastic behavior. At interstory drifts greater than q_{SD} the connection should be assumed to be capable of transmitting 20% of the expected plastic moment capacity of the girder until a total interstory drift angle q_U, obtained from Table 6-1, occurs. At interstory drift angles greater than q_U, the connection should be presumed to have negligible strength.

6.2.1.2 Performance Qualification Data

Table 6-1 presents the applicable performance qualification data for welded unreinforced fully restrained moment-resisting connections, conforming to typical practice prior to the Northridge earthquake.

6.2.2 Simple Shear Tab Connections – with Slabs

The data contained in this section applies to the typical single plate shear tab connection commonly used to connect beams to columns for gravity loads, when moment-resistance is not required by the design, and when concrete slabs are present. Figure 6-2 presents a detail for this connection. It is characterized by rolled wide flange beams connected to either the major or minor axis of wide flange column sections. Beam webs are connected to the column with a single plate shear tab, welded to the column and bolted to the beam web. A concrete floor slab, or slab on metal deck is present at the top flange of the beam.

**Table 6-1 Performance Qualification Data –
Welded Fully Restrained Connection (pre-1994)**

Data Applicability Limits	
Hinge location distance s_h	At distance $d_b/3$ from face of column, unless shear strength of panel zone is less than shear corresponding to development of the flexural strength of beams at the connection, in which case, the hinge location should be taken at the column centerline.
Maximum beam size	Unlimited
Beam material	A36, A572, Gr. 50
Maximum column size	Unlimited
Column steel grades	A36, A572, Gr. 50
Performance Data	
Strength degradation rotation - $q_{SD,}$ radians	0.061-0.0013d_b
Immediate Occupancy rotation - q_{IO}, radians	0.01 radian, but not greater than q_{SD}
Resistance factor, Immediate Occupancy, f	0.8
Collapse Prevention drift angle - q_U – radians	0.053-0.0006d_b
Resistance factor, Collapse Prevention, f	0.8

Notes: d_b= beam depth, inches

Commentary: Although shear tab connections of the type shown in Figure 6-2 are not typically included in design calculations as part of the lateral-force-resisting system, research conducted in support of these Recommended Criteria (FEMA-355D) indicates that they are capable of providing both non-negligible strength and stiffness. Since the typical steel moment-frame structure will have many such connections, the presence of these connections converts the gravity load framing into a highly redundant reserve system to provide additional stiffness and strength for the building after the primary system comprised of fully restrained connected framing has been damaged.

When these connections are loaded such that the top beam flange acts in compression, the slab can act compositely with the beam. When this behavior occurs, the slab will bear against the column and significant moments can *develop through a couple consisting of the slab in compression and the shear tab in tension. This behavior is limited by local crushing of the slab in compression, which behavior initiates at moderate interstory drift angles. Following crushing of the slab, the connections acts as if the slab were not present, and provides relatively modest flexural resistance until very large rotations. Ultimately, at very large rotations, the beam compressive flange will bear against the column, again resulting in development of large moments. Since the beam flange does not crush, this typically results in failure of the shear tab, in tension.*

The criteria for modeling these connections, presented here, neglects the effect of the slab as described above. This is because this behavior occurs only for one direction of loading, and also, because at large deformations, this behavior degrades. However, nothing in this document would preclude more accurate modeling of these connections, that accounts for the slab effects. FEMA-355D provides information that may be useful for this more complex modeling.

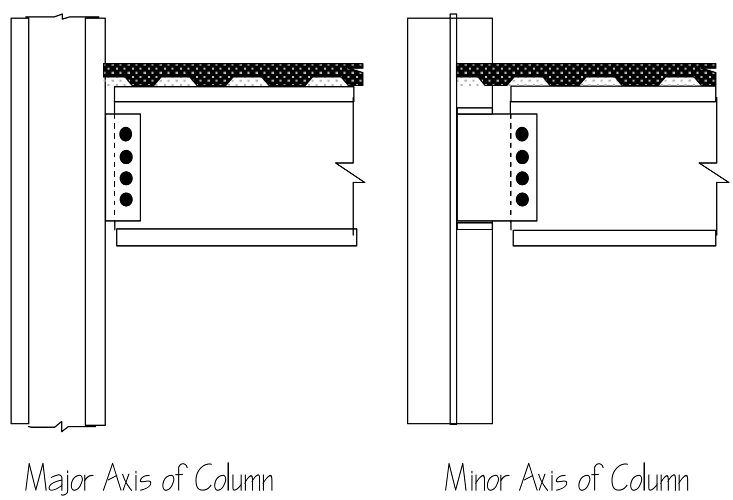


Figure 6-2 Typical Simple Shear Tab Connection with Slab

6.2.2.1 Modeling Assumptions

When included in the analytical model used to predict earthquake induced demands, the stiffness and hysteretic characteristics of framing with simple shear tab connections should be taken in accordance with the recommendations of this section.

6.2.2.1.1 Linear Analysis

The connection stiffness should be explicitly modeled as a rotational spring that connects the beam to the column. The spring stiffness, K_q should be taken as:

$$K_q = 28000(d_{bg} - 5.6) \qquad (6\text{-}1)$$

where d_{bg} is the depth of the bolt group in inches and K_q is in units of k-inches per radian. In lieu of explicit modeling of the connection, beams that frame into columns with simple shear tab connections may be modeled with an equivalent rigidity, EI_{eq} taken as:

$$EI_{eq} = \frac{1}{\dfrac{6h}{l_b^2 K_q} + \dfrac{1}{EI_b}} \qquad (6\text{-}2)$$

where:

E = the modulus of elasticity, kip/square inch
h = the average story height of the columns above and below the beam, inches
I_b = the moment of inertia of the beam, $(\text{inches})^4$
l_b = the beam span center to center of columns, inches

6.2.2.1.2 Nonlinear Analysis

The connection should be explicitly modeled as an elastic-perfectly-plastic rotational spring. The elastic stiffness of the spring should be taken as given by Equation 6-1. The plastic strength of the spring should be determined as the expected plastic moment capacity of the bolt group, calculated as the sum of the expected yield strength of the bolts and their distance from the neutral axis of the bolt group.

6.2.2.2 Performance Qualification Data

Table 6-2 presents the applicable performance qualification data for shear tab connections of beams to columns, with slabs present.

6.2.3 Simple Shear Tab Connections – Without Slabs

The data contained in this section applies to the typical single plate shear tab connection commonly used to connect beams to columns for gravity loads, when moment-resistance is not required by the design and slabs are not present. Figure 6-3 presents a detail for this connection. It is characterized by rolled wide flange beams connected to either the major or minor axis of wide flange column sections. Beam webs are connected to the column with a single plate shear tab, welded to the column and bolted to the beam web. Diaphragms may not be present, and if present consist of wood sheathing, unfilled metal deck, or horizontal steel bracing.

Commentary: Shear tab connections without slabs present behave in a very similar manner to shear tabs with slabs, except that the composite behavior with the slab discussed in the previous section does not occur. Since the modeling criteria for connections with slabs neglect the strength contribution of the slab, the criteria presented herein for connections without slabs are essentially identical to those presented in the previous section.

Table 6-2 Performance Qualification Data – Shear Tab Connections with Slabs

Data Applicability Limits	
Hinge location distance s_h	at center line of bolts
Maximum beam size	Unlimited
Beam material	A36, A572, Gr. 50
Maximum column size	Unlimited
Column steel grades	A36, A572, Gr. 50
Performance Data	
Strength degradation rotation - $q_{SD,}$ radians	0.039-0.0002d_{bg}
Immediate Occupancy rotation - q_{IO}, radians	0.025, but not greater than q_{SD}
Resistance factor, Immediate Occupancy, *f*	0.90
Collapse Prevention drift angle - q_U – radians	0.16-0.0036d_{bg}
Resistance factor, Collapse Prevention, *f*	0.80

Note: d_{bg} = bolt group depth, measured from center of top bolt to center of bottom bolt, inches

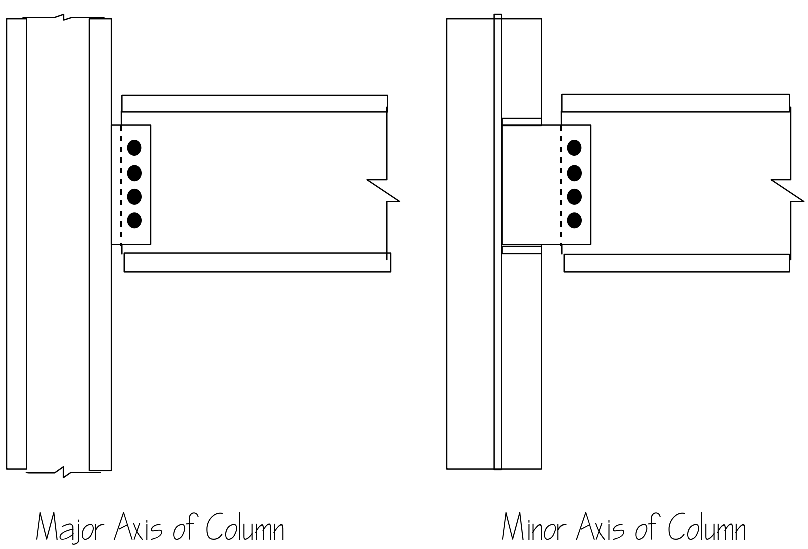


Figure 6-3 Typical Simple Shear Tab Connection Without Slab

6.2.3.1 Modeling Assumptions

Shear tab connections without slabs present should be modeled the same as shear tab connections with slabs present, as indicated in Section 6.2.2.1, except that for nonlinear analysis, performance qualification data shall be as indicated in Table 6-3.

6.2.3.2 Performance Qualification Data

Table 6-3 presents the applicable performance qualification data for shear tab connections of beams to columns, without slabs present.

Table 6-3 Performance Qualification Data – Shear Tab Connections (No Slab)

Data Applicability Limits	
Hinge location distance s_h	At center line of column
Maximum beam size	Unlimited
Beam material	A36, A572, Gr. 50
Maximum column size	Unlimited
Column steel grades	A36, A572, Gr. 50
Performance Data	
Strength degradation rotation - $q_{SD,}$ radians	0.16-0.0036d_{bg}
Immediate Occupancy rotation - q_{IO}, radians	0.030, but not greater than q_{SD}
Resistance factor, Immediate Occupancy, f	0.90
Collapse Prevention drift angle - q_U – radians	0.16-0.0036d_{bg}
Resistance factor, Collapse Prevention, f	0.80

Note: d_{bg} = bolt group depth, measured from center of top bolt to center of bottom bolt, inches

6.3 Basic Design Approach for Connection Upgrades

This section provides recommended criteria on basic principles of connection upgrade design, including selection of an appropriate connection upgrade detail, estimation of locations of inelastic behavior (formation of plastic hinges), determination of probable plastic moment at hinges, determination of shear at the plastic hinge, and determination of design strength demands at critical sections of the assembly. The designer should utilize these basic principles in the calculations for all connection upgrades, unless specifically noted otherwise in these *Recommended Criteria*.

6.3.1 Frame Configuration

Upgraded frames should be proportioned and detailed so that the required drift angle of the frame can be accommodated through elastic deformation and the development of plastic hinges at pre-determined locations within the frame. Figure 6-4 indicates a frame in which inelastic drift is accommodated through the development of plastic flexural deformation (plastic hinges) within the beam span, remote from the face of the column. Such behavior may be obtained by locally stiffening and strengthening type FR connections, using cover plates, haunches and similar detailing, such that the ratio of flexural demand to plastic section capacity is maximum at these interior span locations. Other locations at which plastic deformation may take place in frames, depending on the configuration, detailing, and relative strength of the beams, columns, and connections include: within the connection assembly itself, as is common for shear tab type framing connections, within the column panel zone, or within the column. The total interstory drift angle, as used in these *Recommended Criteria* is equal to the sum of the plastic drift, as described herein, and the elastic interstory drift.

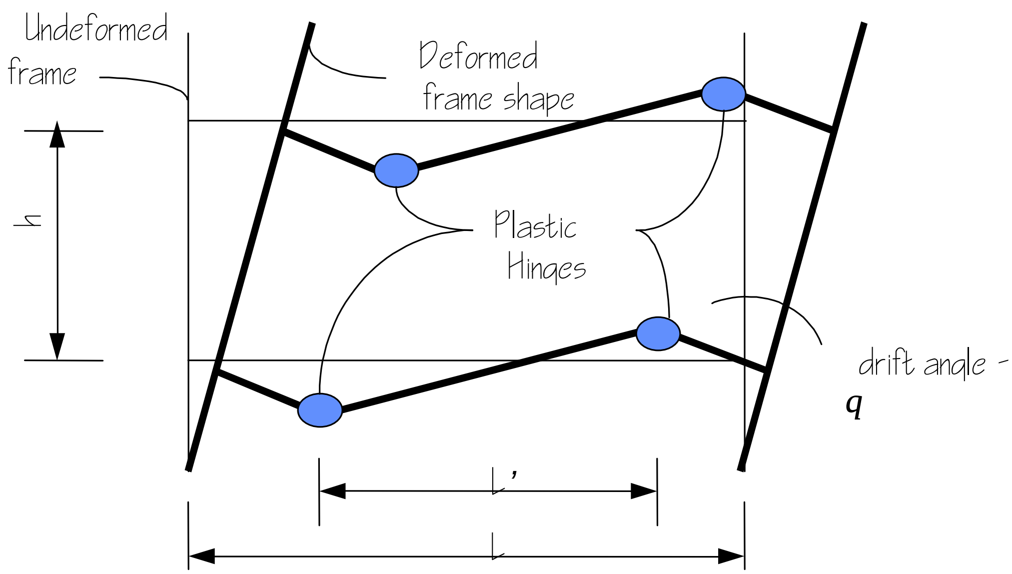


Figure 6-4 Inelastic Behavior of Frames with Hinges in Beam Span

Commentary: Nonlinear deformation of frame structures is accommodated through the development of inelastic flexural or shear strains within discrete regions of the structure. At large inelastic strains these regions can develop into plastic hinges, which can accommodate significant concentrated rotations at constant (or nearly constant) load through yielding at tensile fibers and yielding and buckling at compressive fibers. If a sufficient number of plastic hinges develop in a frame, a mechanism is formed and the frame can deform laterally in a plastic manner. This behavior is accompanied by significant energy dissipation and potentially substantial damage to the highly strained elements. The formation of hinges in columns, as opposed to beams, is generally undesirable, as this may result in the formation of mechanisms with relatively few elements *participating, so called "story mechanisms," and consequently little energy dissipation throughout the structure.*

The prequalified connection contained in the building codes prior to the 1994 Northridge earthquake was based on the development of plastic hinges within the beams at the face of the column, or within the column panel zone. If the plastic hinge develops in the column panel zone, the resulting column deformation results in very large secondary stresses on the beam flange to column flange joint, a condition that can contribute to brittle failure. If the plastic hinge forms in the beam, at the face of the column, this can result in large strain demands on the weld metal and surrounding heat affected zones. These conditions can also lead to brittle failure.

Welded steel moment-frame structures are expected to be capable of extensive amounts of energy dissipation through the development of plastic hinges. In order to achieve reliable performance of these structures, frame configurations should avoid a strong beam-weak column design that can lead to column hinging and story collapse mechanisms. Further, beam-column connections should be configured to force the inelastic action (plastic hinge) away from the column face, where its performance is less dependent on the material and workmanship of the welded joint. This can be done either by local reinforcement of the connection, or local reduction of the cross section of the beam, at a distance away from the connection. Plastic hinges in steel beams have finite length, typically on the order of half the beam depth. Therefore, the location for the plastic hinge should be shifted at least that distance away from the face of the column. When this is done through reinforcement of the connection, the flexural demands on the columns, for a given beam size, are increased. Care must be taken to ensure that weak column conditions are not inadvertently created by local strengthening of the connections.

Many existing WSMF structures were not configured in the original design to produce a strong-column, weak-beam condition. In these structures, connection upgrades that reinforce the beam section locally at the connection, to shift the location of plastic hinging into the beam span, will have little effect, as plastic behavior of the frame will be controlled through plastic hinging of the columns. In such structures, upgrade should include strengthening of the columns with cover plating or other similar measures, or alternatively, the provision of supplemental lateral force resisting elements such as braced frames or shear walls. Upgrade recommendations are discussed in Chapter 5.

Connection upgrades of the type described above, while believed to be effective in preventing brittle connection fractures, will not prevent structural damage from occurring. Brittle connection fractures are undesirable for several reasons. First, severe connection degradation can result in loss of gravity load carrying capacity of the framing at the connection and the potential development of local collapse. From a global perspective, the occurrence of many connection *fractures results in a substantial reduction in the lateral-force-resisting strength and stiffness of the structure which, in extreme cases, can result in instability and collapse. Connections upgraded as described in this document should experience many fewer brittle fractures than unmodified connections. However, the formation of a plastic hinge within the beam is not a completely benign event. Beams that have experienced significant plastic rotation at such hinges may exhibit large buckling and yielding deformation, as well as concurrent localized damage to floor slabs and other supported elements. In severe cases, this damage must be repaired. The cost and difficulty of such repairs could be comparable to the costs incurred in repairing connection fracture damage of the types experienced in the Northridge earthquake. The primary difference is that life safety protection will be significantly enhanced and most upgraded structures should continue to be safe for occupancy, while repairs are made.*

If the types of damage described above are unacceptable for a given building, then alternative upgrade systems should be considered, which will reduce the plastic deformation demands on the structure during a strong earthquake. Appropriate methods of achieving such goals include the installation of supplemental braced frames, shear walls, energy dissipation systems, base isolation systems, and similar structural systems.

6.3.2 Required Drift Angle Capacity

For systematic upgrade design, the required drift angle capacity of connection assemblies should be sufficient to withstand the total (elastic and plastic) interstory drift likely to be induced in the frame by earthquake ground shaking, as predicted by analysis, while providing sufficient confidence with regard to achievement of the desired performance, in accordance with the procedures of Chapter 3. Section 6.6 provides data on the drift angle capacity of several prequalified connection upgrade details, together with design guidelines for these connection upgrades and limits on the applicability of the prequalification. Section 6.7 provides performance data for several types of moment-resisting connections that have been prequalified for use in new steel moment-frame construction. Section 6.8 provides descriptive information on several types of proprietary connection technologies that may be considered for seismic upgrade applications. Section 6.9 provides recommended criteria for determining the factored drift angle capacity of connection upgrades that are not prequalified.

For the purposes of Simplified Upgrade, frames shall be classified either as Ordinary Moment Frames (OMF) or Special Moment Frames (SMF) and connection upgrade details that are prequalified for the appropriate system, as indicated in Section 6.6 of these guidelines, should be selected. For purposes of simplified upgrades, a frame should be considered an SMF system if the construction documents indicate it was designed as a Special Moment Resisting Frame, a Ductile Moment Resisting Frame, or if the original design documents indicate that any of the design values indicated in the column labeled “SMF” in Table 6-4 were used in determining the design seismic forces for the frame in the original design. A frame should be considered an OMF if the design documents indicate it was designed as an OMF or if any of the design values indicated in the column labeled "OMF" in Table 6-4 were used in determining the design seismic forces for the frame in the original design. If sufficient documentation is not available to permit determination of the original intended system for the structure, an SMF should be assumed.

Table 6-4 Design Coefficients for SMF and OMF Systems

Design Coefficient	OMF	SMF
K (buildings designed to 1985 or earlier edition of UBC, or 1990 or earlier editions of BOCA or SSBC.)	1.0	0.67
R_w (buildings designed to UBC editions 1988 - 1994)	6	12
R (buildings designed to 1997 UBC, or 1993 or later editions of BOCA or SSBC.)	4	8

Commentary: In Systematic Upgrades, a complete analysis of the structure is performed, in accordance with the criteria of Chapter 3. In this analysis, an estimate is developed of the forces and deformations induced by response to earthquake ground shaking, and based on these estimated forces and deformations, and the estimated capacity of the frame and its individual components to resist these demands, a level of confidence with regard to the ability of the frame to provide desired performance is estimated.

In Simplified Upgrades, performance evaluation of the structure, in accordance with Chapter 3, is not performed. Rather than providing a specific level of confidence that the structure is capable of a particular performance, simplified upgrades are intended only to provide the structure with the level of reliability implicitly presumed by the code provisions under which it was originally designed. Until recently, the building codes only recognized two types of moment-resisting steel frame systems: a system with significant intended inelastic response capability called either a Special Moment Frame, or in some codes, a Ductile Moment-Resisting Frame; and frames having only limited inelastic response capability, typically called an Ordinary Moment Frame.

Table 6-4 classifies framing systems, using the terminology contained in the 1997 NEHRP Recommended Provisions for New Buildings and 1997 AISC Seismic Design Specification, as either an SMF or an OMF.

In addition to these two categories of moment-resisting frames, some steel moment-resisting frames are part of a dual structural system, in which the frames provide a secondary system of lateral-force resistance for a primary system *comprised of braced frames or shear walls. Upgrade of such structures, using the Simplified procedure is not recommended.*

6.3.3 Connection Configuration

For Simplified Upgrade, a connection upgrade configuration should be selected that is compatible with the appropriate structural system. No further qualification of the design is necessary, other than to ensure that the connection configuration does not create any of the following conditions, as defined in the building code, or make an existing such condition more severe:

a. Weak column - strong beam

b. Weak story

c. Soft story

d. Torsional Irregularity

For Systematic Upgrade, a connection configuration that is capable of providing sufficient factored drift angle capacity to provide a suitable level of confidence should be selected. Section 6.6 presents data on a series of prequalified connection upgrade details, from which an appropriate detail may be selected. These connection upgrades details are prequalified for use within certain ranges of member sizes and frame configuration. If these connection upgrade details are to be employed outside the range of applicability, project specific connection qualification should be performed. If project-specific connection qualification is to be performed, a connection of any configuration may be selected and qualified for acceptability using the procedures of Section 6.9.

6.3.4 Determine Plastic Hinge Locations

Based on the data presented in these *Recommended Criteria* for prequalified connection upgrades, or data obtained from a qualification testing program for configurations that are qualified on a project specific basis, the location of expected plastic hinge formation, s_h, as indicated in Figure 6-5 should be identified. The plastic hinge locations presented for prequalified connection upgrades are valid for beams with gravity loads representing a small portion of the total flexural demand and for conditions of strong column, weak beam. For frames in which gravity loading produces significant flexural stresses in the members, or frames that do not have strong-column, weak-beam configurations, locations of plastic hinge formation should be determined based on methods of plastic analysis.

Commentary: The suggested location for the plastic hinge, as indicated by the parameter s_h in the prequalification data, is valid only for frames with limited gravity loading present on the frame beams, or for frames in which yielding will actually occur in the beam, rather than in the column panel zone or the column itself. If significant gravity load is present, or if panel zones or columns are the weak links in the frame, this can shift the locations of the plastic hinges, and in *the extreme case, change the form of the collapse mechanism. If flexural demand on the girder due to gravity load is less than about 30% of the girder plastic capacity, this effect can safely be neglected, and the plastic hinge locations taken as indicated, as long as beam flexure, rather than panel zone shear, column flexure, or beam shear is the dominant inelastic behavior for the frame. If gravity demands significantly exceed this level then plastic analysis of the girder should be performed to determine the appropriate hinge locations.*

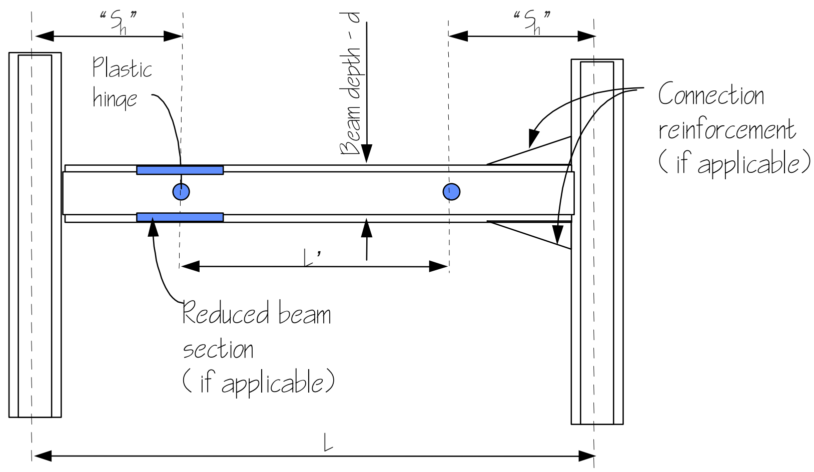


Figure 6-5 Location of Plastic Hinge Formation

6.3.5 Determine Probable Plastic Moment at Hinges

For fully restrained connections designed to develop plastic hinging in the beam or girder, the probable plastic moment at the location of the plastic hinge should be determined as:

$$M_{pr} = C_{pr} R_y Z_e F_y \tag{6-3}$$

where:

M_{pr} = Probable peak plastic hinge moment.

C_{pr} = A factor to account for the peak connection strength, including strain hardening, local restraint, additional reinforcement, and other connection conditions. For most connection types, C_{pr} is given by the formula:

$$C_{pr} = \frac{F_y + F_u}{2F_y} \tag{6-4}$$

A value of 1.2 may be used for all cases, except where otherwise noted in the individual connection design procedures included with the prequalifications in later sections of these *Recommended Criteria*.

R_y = A coefficient, applicable to the beam or girder material, obtained from the *AISC Seismic Provisions*

Z_e = The effective plastic modulus of the section (or connection) at the location of the plastic hinge.
F_y = the specified minimum yield stress of the material of the yielding element.
F_u = the specified minimum tensile stress of the material of the yielding element.

For connections that do not develop plastic hinges in the beam, the hinge strength should be calculated, or determined from tests, for the pertinent yield mechanism, considering the variation in material properties of the yielding elements. For prequalified connection upgrades and connections, calculation methods to determine the yield strengths of the various active mechanisms are given in the design procedure accompanying the individual prequalification.

Commentary: The AISC Seismic Provisions use the formulation $1.1R_yM_p+M_v$ for calculation of the quantity SM^*_{pb}, which is used in calculations for column strength (strong-column, weak-beam), and for required shear strength of panel zones. As described in FEMA-355D, research has shown that, for most connection types, the peak moment developed is somewhat higher than the 1.1 factor would indicate. Therefore, for these guidelines, the factor C_{pr}, calculated as shown, is used for individual connections, with a default value of 1.2 applicable to most cases.

6.3.6 Determine Shear at the Plastic Hinge

The shear at the plastic hinge should be determined by statics, considering gravity loads acting on the beam. A free body diagram of that portion of the beam between plastic hinges is a useful tool for obtaining the shear at each plastic hinge. Figure 6-6 provides an example of such a calculation. For the purposes of such calculations, gravity load should be based on the load combinations indicated in Section 6.5.1.

6.3.7 Determine Strength Demands at Each Critical Section

In order to complete the design of the connection upgrade, including, for example, sizing the various plates, bolts, and joining welds, which make up the connection, it is necessary to determine the shear and flexural strength demands at each critical section. These demands may be calculated by taking a free body of that portion of the connection assembly located between the critical section and the plastic hinge. Figure 6-7 demonstrates this procedure for two critical sections for the beam shown in Figure 6-6.

Commentary: Each unique connection configuration may have different critical sections. The vertical plane that passes through the joint between the beam flanges and column (if such joining occurs) will typically define at least one such critical section, used for designing the joint of the beam flanges to the column, as well as evaluating shear demands on the column panel zone. A second critical section occurs at the center line of the column. Moments calculated at this point are used to check strong-column, weak-beam conditions. Other critical sections should be selected as appropriate.

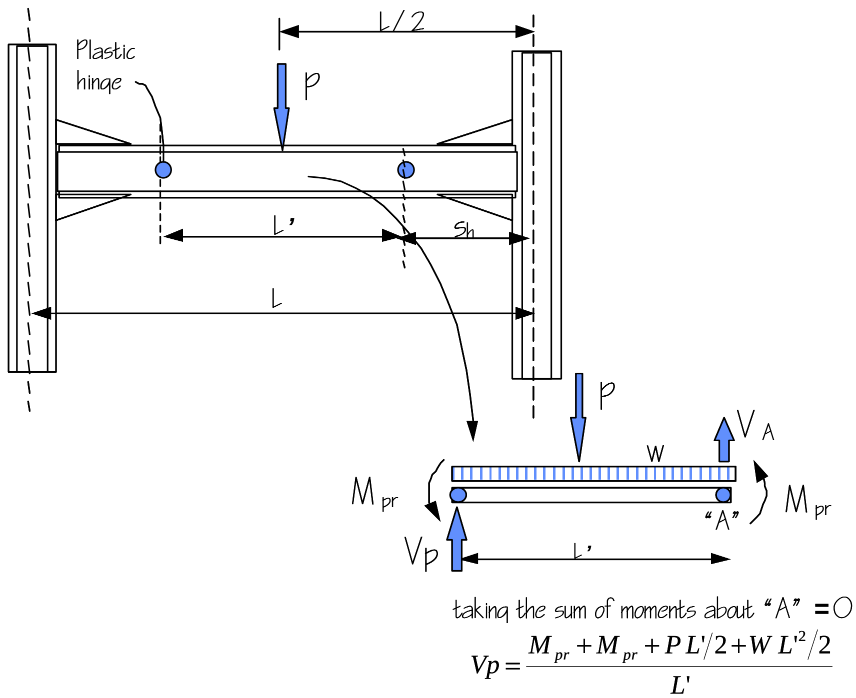


Figure 6-6 Sample Calculation of Shear at Plastic Hinge

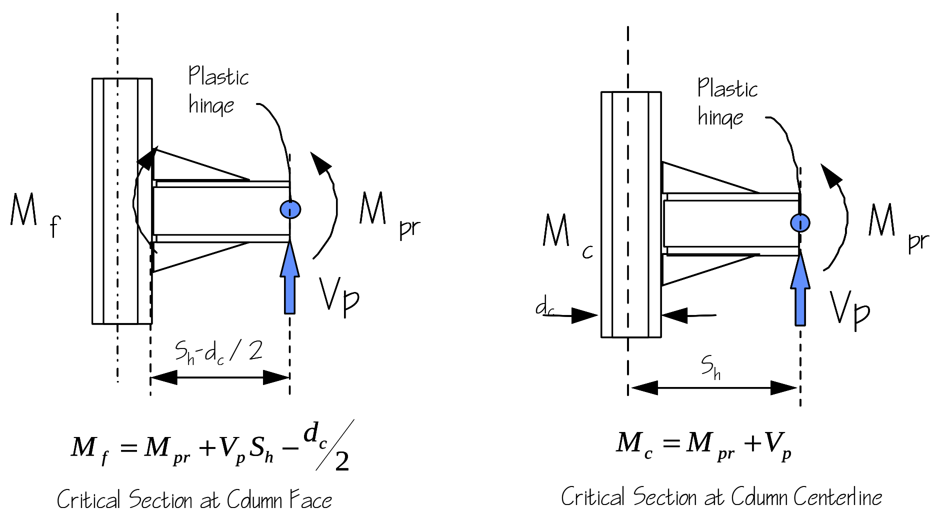


Figure 6-7 Calculation of Demands at Critical Sections

6.3.8 Yield Moment

The design procedures for some prequalified connections contained in these *Recommended Criteria* require that the moment at the face of the column at onset of plastic hinge formation, M_{yf}, be determined. M_{yf} may be determined from the following equation:

$$M_{yf} = C_y M_f \tag{6-5}$$

where:

$$C_y = \frac{1}{C_{pr}\frac{Z_{be}}{S_b}} \tag{6-6}$$

C_{pr} = the peak connection strength coefficient defined in Section 6.3.5
S_b = the elastic section modulus of the beam at the zone of plastic hinging
Z_{be} = the effective plastic section modulus of the beam at the zone of plastic hinging.

6.4 General Requirements

This section provides criteria for connection upgrade design conditions that are considered to be general, that is, those conditions which, when they occur in a connection upgrade, are considered to perform in a similar way, or at least to have the same requirements for successful performance, irrespective of the connection type being used. The designer should employ these criteria in the design of all connection types, except when specific testing has been performed that qualifies the connection for use with different conditions, or unless otherwise specifically indicated in these *Recommended Criteria*.

6.4.1 Framing

6.4.1.1 Beam and Column Strength Ratio

For multistory SMF systems, frames should be configured with a strong-column, weak-beam configuration, to avoid the formation of single-story mechanisms. As a minimum, Equation 9-3 of *AISC Seismic Provisions* should be satisfied. In the application of Equation 9-3, the quantity M_c as defined in Section 6.3.7 of these *Recommended Criteria* should be substituted for the quantity M^*_{pb}.

Commentary: When subjected to strong ground shaking, multi-story structures with columns that are weaker in flexure than the attached beams can form single story mechanisms, in which plastic hinges form at the base and top of all columns in a story. Once such a mechanism forms in a structure, nearly all of the earthquake induced lateral displacement will occur within the yielded story, which can lead to very large local drifts and the onset of P-D instability and collapse.

Building codes permitted frames to be designed with weak-column, strong-beam configurations until 1988. Therefore, many existing steel moment-frame buildings have such configuration. Further, some types of connection upgrades, through local strengthening of the beam ends, have the potential to create weak-

column, strong-beam systems in frames that originally did not have such configuration. Although weak-column, strong-beam designs are not desirable, AISC Seismic does permit their use under certain conditions, even for SMF systems. Before utilizing weak-column, strong-beam configurations, designers should be aware that the prequalified connections for SMF systems contained in these Recommended Criteria are based on tests using strong columns.

Nonlinear analyses of representative frames have clearly shown that the use of the provisions described above will not completely prevent plastic hinging of columns. This is because the point of inflection in the column may move away from the assumed location at the column mid-height once inelastic beam hinging occurs, and because of global bending induced by the deflected shape of the building, of which the column is a part.

Except for the case when a column hinge mechanism forms, column hinging is not a big problem, provided that the columns are designed as compact sections, are properly braced and axial loads are not too high. It is well understood that a column hinge will form at the base of columns that are continuous into a basement, or that are rigidly attached to a stiff and strong foundation.

6.4.1.2 Beam Flange Stability

Beam flange slenderness ratios $b_f/2t_f$ (b/t) should be limited to a maximum value of $52/\sqrt{F_y}$, as required by *AISC Seismic Provisions*. For moment frame beams with Reduced Beam Section (RBS) connections, it is recommended that the $b_f/2t_f$ be determined based on the flange width b_f measured at the ends of the center 2/3 of the reduced section of the beam unless gravity loads are large enough to shift the hinge point significantly from the center point of the reduced section.

Commentary: The AISC Seismic Provisions require that beam flange slenderness ratios $b_f/2t_f$ (b/t) be limited to a maximum of 52/ÖF_y. This specific value is intended to allow some plastic rotation of the beam to occur before the onset of local buckling of the flanges, a highly undesirable phenomenon. Widespread buckling of beam flanges in a moment resisting frame can result in development of frame strength degradation increasing both story drifts and the severity of P-D effects and therefore should be avoided. Local flange buckling results in very large local straining of the flanges and the early on-set of low-cycle fatigue induced tearing of the beam flanges, which ultimately limits the ability of the assembly to withstand cyclic inelastic rotation demands. Further, severely buckled beam flanges can be even more difficult to repair than fractured beam connections.

Notwithstanding the above, under large plastic rotation demands, buckling of beam flanges will inevitably occur. The value of the b/t of the beam involved in a specific connection can have a major effect on how the beam column assembly performs. Beams and girders used in moment frames should comply with the *limits specified by the AISC Seismic Provisions, except as specifically modified by individual connection prequalifications or qualification tests.*

6.4.1.3 Beam Web Stability

Moment-frame beams should be selected that have web height-to-thickness ratios, h_c/t_w of not greater than $418/\sqrt{F_y}$.

Commentary: The AISC Seismic Provisions permits use of beams with web h_c/t_w up to as high as 520/ÖF_y, for beams without axial load. Most of the testing under this project has been conducted on beams such as W30x99 and W36x150, both of which barely conform to $h_c/t_w \leq$ 418/ÖF_y. Since many of the specimens exhibited significant web buckling in the area of plastic hinges, it is not considered prudent to utilize beams with relatively thinner webs in moment frames. Although stiffening of the webs could be done to limit web buckling, it is possible that stiffeners could be detrimental to connection performance. Since connections with web stiffeners were not tested, such connections have not been prequalified. See FEMA-355D, State of the Art Report on Connection Performance, for further discussion of web buckling of moment-frame beams.

6.4.1.4 Beam Span and Depth Effects

The performance of moment-resisting beam-column connections is strongly related both to beam depth and beam span-to-depth ratio. Data accompanying each of the prequalified connection upgrades presented in Section 6.6 includes specification of maximum beam depths and minimum beam span-to-depth ratio. Connection upgrade details presented in Section 6.6 should not be used for cases where beam depth exceeds the indicated limit unless project-specific qualification, in accordance with Section 6.9 is performed. For Simplified Upgrade, connection upgrade details should not be used in cases where the beam span-to-depth ratio is less than the indicated amount unless project-specific qualification, in accordance with Section 6.9, is performed. For Systematic Upgrade, connection upgrade details may be used on beams with spans that have smaller span-to-depth ratio than the limiting value indicated in the prequalification provided that the acceptance criteria used in performance evaluation for interstory drift capacity q as limited by local connection behavior is modified as indicated by the equation:

$$q' = \frac{8d}{L}\left(1 + \frac{L - L'}{L}\right)q \tag{6-7}$$

where:

q' = the median interstory drift angle capacity for connection behavior for beams with small span-to-depth ratio

q = the median interstory drift angle capacity listed in the prequalification for connection behavior for beams meeting the span to depth limitations of the prequalification

L = the span of the beam, center-line-to-center-line of columns, inches

L' = the effective span of the beam between plastic hinge locations, inches

d = the beam depth in inches

Where the effective span L' of the beam between points of plastic hinging, is such that shear yielding of the beam will occur, rather than flexural yielding, the web of the beam should be stiffened between the points of plastic hinging, and braced as required by the *1997 AISC Seismic Provisions* for long links in eccentric braced frames.

Commentary: Both beam depth and beam span-to-depth ratio are significant in the inelastic behavior of beam-column connections. At a given induced curvature, deep beams will undergo greater straining than shallower beams. Similarly, beams with shorter span-to-depth ratio will have a sharper moment gradient across the beam span, resulting in reduced length of the beam participating in plastic hinging and increased strains under inelastic rotation demands. Most of the beam-column assemblies tested under this project used configurations approximating beam spans of about 25 feet and beam depths varying from W30 to W36 so that beam span-to-depth ratios were typically in the range of 8 to 10. Equation 6-7 approximately accounts for these effects. Additional information may be found in FEMA-355D, State of the Art Report on Connection Performance.

6.4.1.5 Beam Flange Thickness Effects

The connection upgrade prequalifications contained in these *Recommended Criteria* are limited in application to specific beam flange thicknesses. These limitations are noted in the tabulated data for each connection. For frames designed using project-specific connection qualifications, connection tests used in the connection qualification program should employ beam flanges of similar or greater thickness than those used in the frame.

Commentary: In addition to controlling the stability of the flange under compressive loading, as described above, beam flange thickness also affects the size of welds in welded connections. Although it is not a given that larger welds will be less reliable than smaller welds, greater control may be necessary to ensure their performance, and quality control may be more difficult. Additionally, residual stresses are likely to be higher in thicker material with thicker welds.

6.4.1.6 Lateral Bracing at Beam Flanges at Plastic Hinges

Plastic hinge locations that are remote from the column face in beams that do not support a slab should be provided with supplemental bracing, as required by the 1997 *AISC Seismic Provisions*. Where the beam supports a slab and is in direct contact with the slab along its span length, supplemental bracing need not be provided.

Commentary: The 1997 AISC Seismic Provisions require that beam flanges be braced at plastic hinge locations. Because plastic hinges have been moved away from the column face for some of the connection upgrade types in this section, a strict interpretation of the provisions would lead to a requirement that flanges at such hinges be laterally braced. Limited testing conducted as part of this project (FEMA-355D) suggests that, as long as the hinging beam is connected to a concrete slab, excessive strength deterioration due to lateral buckling will not occur within the ranges of drift angle normally considered important. Therefore, these Recommended Criteria do not require supplemental bracing of plastic hinge locations adjacent to column connections of beams supporting slabs.

For those cases where supplemental bracing of beam flanges near plastic hinges is appropriate, great care must be taken in detailing and installation of such bracing to ensure that attachments are not made directly within the area of anticipated plastic behavior. This is because of the inherent risk of reducing plastic deformation capacity for the beam by introducing stress concentrations or metallurgical notches into the region of the beam that must undergo plastic straining. See FEMA-355D, State of the Art Report on Connection Performance, for further discussion of flange bracing.

6.4.1.7 Welded Shear Studs

Welded shear studs, or other attachments for composite action with slabs or for diaphragm shear transfer, should not be installed within the hinging area of moment-frame beams. The hinging area is defined as the distance from the column flange face to one half the beam depth beyond the theoretical hinge point. Standard arc-spot weld attachments may be made in the hinging area, but shot-in, or screwed attachments should not be permitted.

Commentary: It has been shown in some tests that welded shear studs and the rapid increase of section caused by composite action can lead to beam flange fractures when they occur in the area of the beam flange that is undergoing large cyclic strains. It is not certain whether the welding of the studs, the composite action, or a combination of the two is the cause, but, based on the limited evidence, it is judged to be prudent to permit no studs in the hinging area. It is also prudent to permit no attachments that involve penetration of the flanges in the hinging region.

6.4.2 Welded Joints

6.4.2.1 Through-Thickness Strength

The through-thickness strength demands on existing column material should be limited to the values given in Table 6-5. Through-thickness demands should be calculated as the applied flange force, divided by the projected area of the welded joint on the column flange, using the procedures of Section 6.3.7 to calculate the applied force at this critical section.

Table 6-5 Column Flange Through-Thickness Strength

Column Flange Material Specification	$F_{t\text{-}t}$
Hot rolled wide flange columns conforming to A36, ASTM A572 Grade 50, or ASTM A992, or ASTM A913 rolled later than 1994 and having sulfur content not in excess of 0.05% by weight.	No limit
All other material	$0.8F_u$

Commentary: Early investigations of connection fractures in the 1994 Northridge earthquake identified a number of fractures (types C3 and C5 Section 2.3.2) that appeared to be the result of inadequate through-thickness strength of the column flange material. As a result of this, in the period immediately following the Northridge earthquake, a number of recommendations were promulgated that suggested limiting the value of through thickness stress demand on column flanges to a value of 40 ksi, applied to the projected area of the beam flange attachment. This value was selected to ensure that through-thickness yielding did not initiate in the column flanges of FR connections and often controlled the overall design of a connection subassembly.

It is important to prevent the inelastic behavior of connections from being controlled by through-thickness yielding of column flanges. This is because it would be necessary to develop very large local ductilities in the column flange material in order to accommodate even modest plastic rotation demands on the assembly. However, the actual cause for the type C3 fractures, that were initially identified as through-thickness failures of the column flange are now believed to be unrelated to this material property. Rather, it appears that C3 damage occurred when fractures initiated in defects present in the complete joint penetration (CJP) weld root, not in the flange material (FEMA-355E). These defects sometimes initiated a crack, that under certain conditions, propagated into the column flange, giving the appearance of a through-thickness failure. Detailed fracture mechanics investigations conducted under this project confirm that the C3 damage initially identified as through-thickness failures are likely to have occurred as a result of certain combinations of material strength and notch toughness, conditions of stress in the connection, and the presence of critical flaws in the welded joint.

As part of the research conducted in support of the development of these Recommended Criteria, extensive through-thickness testing of modern steels, meeting the ASTM A572, Gr. 50 and ASTM A913, Gr. 65 specifications has been conducted to determine the susceptibility of modern column materials to through-thickness failures (FEMA 355A, State of the Art Report on Base Metals and Fracture). This combined analytical and laboratory research clearly showed that due to the restraint inherent in welded beam flange to column flange joints, the through thickness yield and ultimate strengths of the column material is significantly elevated in the region of the connection. Further, for the modern materials tested, these strengths significantly exceed those that can be delivered to the column by beam material conforming to these same specifications. For this reason, no limits are suggested for the through-thickness strength of modern steel materials with controlled sulfur contents, as required by the FEMA-353 Recommended Specifications and Quality Assurance Guidelines for Steel Moment-Frame Construction for Seismic Applications.

Notwithstanding the above, it is known that in the past, lamellar tearing of thick column flanges occasionally occurred during the fabrication and erection process. This lamellar tearing was a result of high through thickness strains induced by welding on material that had excessive sulfur inclusions. These sulfur inclusions, which were flattened and elongated during the shape rolling process could form planes of weakness within the shape that were susceptible to this tearing. It is known that steel with relatively high sulfur content is more susceptible to this behavior than shapes with lower sulfur contents. Also, it is known that shapes that undergo a significant amount of working during the rolling process are more susceptible as well, as the rolling process tends to flatten the sulfide inclusions and align them in the rolling direction. Modern steel production often uses a continuous casting process in which the steel is cast in a shape that is near that of the final product, resulting in the sulfur being uniformly distributed throughout the shape and therefore less susceptibility to lamellar tearing.

Table 6-5 recommends a limit of $0.8F_u$ *for through-thickness stress on older steels, that may be susceptible to through-thickness tearing, based on a statistical survey of the relationship of through-thickness strength to longitudinal strength for structural steels (Barsom, 1996).*

6.4.2.2 Base Material Toughness

Material in rolled shapes with flanges 1-1/2 inches or thicker, and sections made from plates that are 2 inches or thicker, should be required to have minimum Charpy V-notch toughness of 20 ft-lbs, at 70 degrees F. Refer to *FEMA-353, Recommended Specifications and Quality Assurance Guidelines for Steel Moment-Frame Construction for Seismic Applications.*

Commentary: The 1997 AISC Seismic Provisions specified minimum notch toughness for rolled shapes with flanges 1-1/2 inches thick or thicker, and sections made from plates 1-1/2 inches thick or thicker, be checked for notch toughness. These Recommended Criteria relax the requirement for toughness of plate material to apply to plates 2 inches or thicker as this was the original intent of the AISC specification, and it is believed that the AISC document will be revised to this requirement.

Research has not clearly demonstrated the need for a specific value of base metal toughness. However, it is judged that base metal notch toughness is important to prevention of brittle fracture of the base metal in the highly stressed areas of the connection. A number of connection assemblies that have been tested have demonstrated base metal fractures at weld access holes and at other discontinuities such as at the ends of cover plates. In at least some of these tests, the fractures initiated in zones of low notch toughness. Tests have not been conducted to determine if higher base metal notch toughness would have reduced the incidence of such fractures.

The Charpy V-Notch (CVN) value of 20 ft.-lbs. at 70 degrees F, recommended here, was chosen because it is usually achieved by modern steels, and because steels meeting this criterion have been used in connections which have performed successfully. Current studies (FEMA 355A, State of the Art Report on Base Metals and Fracture) have indicated that rolled shapes produced from modern steels meet this requirement almost routinely even in the thicker shapes currently requiring testing. It has been suggested that the requirement for this testing could be eliminated and replaced by a certification program administered by the mills. However, such a program is not currently in existence. Until such time as such a certification program is in place, or a statistically meaningful sampling from all major mills has been evaluated, it is recommended that the AISC requirement for testing be continued. According to the Commentary to the 1997 AISC Seismic Provisions, thinner sections are judged not to require testing because they "are generally subjected to enough cross-sectional reduction during the rolling process that the resulting notch toughness will exceed that required." In other words, the notch toughness is required, but testing to verify it on a project basis is not judged to be necessary as it is routinely achieved.

No specific notch toughness requirements are specified for existing materials in steel moment frames. This is because testing of the notch toughness of these materials is costly and difficult and also because there is no practical way to improve the notch toughness of an existing material, other than to replace it. The importance of base material notch toughness with regard to steel moment-frame behavior is not clear, however. High material notch toughness is beneficial in preventing the propagation of minor fractures and flaws into unstable brittle fractures, when such defects are present. However, base metals typically are free *of such defects and therefore, less susceptible to the initiation of the brittle fractures that material notch toughness is effective in preventing.*

6.4.2.3 k-Area Properties

The k-area of rolled wide-flange shapes, which may be considered to extend from the mid-point of the radius of the fillet from the flange into the web, approximately 1 to 1-1/2 inches beyond the point of tangency between the fillet and web, as defined in Figure C-6.1 of the *AISC Seismic Provisions*, is likely to have low toughness and may therefore be prone to cracking caused by welding operations. Designers should detail welds of continuity plates and web doubler plates in columns in such a way as to avoid welding directly in the k-area. Refer to Section 6.4.3 for more information.

Fabricators should exercise special care when making welds in, or near to, the k-area. Where welding in the k-area of columns cannot be avoided, special nondestructive testing is recommended. Refer to *FEMA-353, Recommended Specifications and Quality Assurance Guidelines for Steel Moment-Frame Construction for Seismic Applications.*

Commentary: Recent studies, instigated in response to fabrication problems, have shown that, for rotary-straightened W-shapes, an area of low material toughness can occur in the region of the web immediately adjacent to the flange. In some instances, cracking has occurred in these areas during welding. The Commentary to the AISC Seismic Provisions provides a figure (Fig. C-6.1) that defines the k-area.

The low toughness of the k-area seems to be associated only with rotary-straightened sections. Which sections are rotary straightened varies among the mills. One major domestic supplier rotary-straightens all shapes weighing less than 150 pounds per linear foot. Larger sections are often straightened by other means that do not result in as much loss of toughness in the k-area. Because rolling practice is frequently changed, it is prudent to assume that all rolled sections are rotary-straightened.

6.4.2.4 Weld Filler Metal Matching and Overmatching

The use of weld filler metals and welding procedures that will produce welds with matching or slightly overmatching tensile strength relative to the connected steel is recommended. Welding consumables specified for Complete Joint Penetration (CJP) groove welds of beam flanges and flange reinforcements should have yield and ultimate strengths at least slightly higher than the expected values of yield and ultimate strength of the beam or girder flanges being welded. Significant overmatching of the weld metal should not be required unless overmatching is specified in the connection prequalification or is used in the prototypes tested for project-specific qualification of the connection being used. Flux Cored Arc Welding and Shielded Metal Arc Welding electrodes commonly used in structural construction and conforming to the E70 specifications provide adequate overmatching properties for structural steels conforming to ASTM A36, A572, Grades 42 and 50, A913, Grade 50 and A992. Welded splices of columns of

A913-Grade 65 steel should be made with electrodes capable of depositing weld metal with a minimum ultimate tensile strength of 80 ksi.

Commentary: Undermatched weld metals, that is, weld metals with lower strength than the connected base metals, are beneficial in some applications in that they tend to limit the residual stress state in the completed joint. However, in applications where yield level stresses are anticipated, it is desirable to minimize the amount of plasticity in the welded joint. This can be achieved by employing balanced, or slightly overmatched weld filler metals. The majority of the successful connection tests have used weld metals with yield and tensile strengths in the range of 58 and 70 ksi respectively, which provide matching to moderate overmatching with beams of Grade 50 steel. For additional information refer to FEMA-355B, State of the Art Report on Welding and Inspection.

6.4.2.5 Weld Metal Toughness

For structures in which the steel frame is normally enclosed and maintained at a temperature of 50°F or higher, critical welded joints in seismic force resisting systems, including complete joint penetration (CJP) groove welds of beam flanges to column flanges, CJP welds of shear tabs and beam webs to column flanges, column splices, and similar joints, should be made with weld filler metal providing CVN toughness of 20ft-lbs at -20° F and 40ft-lbs at 70° F and meeting the Supplemental Toughness Requirements for Welding Materials in *FEMA-353 – Recommended Specifications and Quality Assurance Guidelines for Steel Moment-Frame Construction for Seismic Applications*. For structures with lower service temperatures than 50°F, qualification temperatures should be reduced accordingly.

Commentary: Principles of fracture mechanics demonstrate the importance of notch toughness to resist fracture propagation from flaws, cracks, and backing bars or other stress concentrations, which may be preexisting or inherent, or which may be caused by applied or residual stresses. The 1997 AISC Seismic Provisions requires the use of welding consumables with a rated Charpy V-Notch (CVN) toughness of 20 ft.-lbs. at -20° F, for CJP groove welds used in the Seismic Force Resisting System. Seismic Provisions for Structural Steel Buildings (1997) Supplement No. 1, February 15, 1999, (AISC, 1999) changes this requirement to include "all welds used in primary members and connections in the Seismic Force Resisting System". The rating of the weld filler metal is as determined by the American Welding Society classification or manufacturer certification.

Studies conducted under this project have indicated that not all weld consumables that are rated for 20 ft-lbs of toughness at –20°F will provide adequate toughness at anticipated service temperatures. The supplemental toughness requirements contained in FEMA-353 are recommended to ensure that weld metal of adequate toughness is obtained in critical joints. Most of the beam-column connection tests conducted under this project were made with weld filler metal conforming to either the E70T6 or E70TGK2 designations. These filler *metals generally conform to the recommended toughness requirements. Other weld filler metals may also comply.*

6.4.2.6 Weld Backing, Weld Tabs, and other Welding Details

Weld backing and runoff tabs should be removed from complete joint penetration flange welds, unless otherwise noted in the connection prequalification or demonstrated as not required by project-specific qualification testing. Refer to *FEMA-353, Recommended Specifications and Quality Assurance Guidelines for Steel Moment-Frame Construction for Seismic Applications,* for special requirements for weld backing, weld tabs, and other welding details for moment frame joints. It is not recommended that backing and runoff tabs be removed from existing connections in buildings, unless other upgrades or modifications of the affected connections are being made, in which case such removal is recommended.

The following general procedures may be considered for backing removal. Steel backing may be removed either by grinding or by the use of air arc or oxy-fuel cutting. The zone just beyond the theoretical 90-degree intersection of the beam-to-column flange should be removed either by air arc or oxy-fuel cutting followed by a thin grinding disk, or by a grinding disk alone. This shallow gouged depth of weld and base metal should then be tested by magnetic particle testing (MT) to determine if any linear indications remain. If the area is free of indications the area may then be re-welded. The preheat should be maintained and monitored throughout the process. If no further modification is to be made or if the modification will not be affected by a reinforcing fillet weld, the reinforcing fillet may be welded while the connection remains at or above the minimum preheat temperature and below the maximum interpass temperature.

Commentary: It was originally hypothesized, following the 1994 Northridge earthquake that weld backing created an effective crack equal to the thickness of the backing and that this phenomena was responsible for many of the fractures that had occurred. Finite-element analyses of welded joints (Chi, et al., 1997) have shown that although the backing does create some notch effect, a far more significant factor is the fact that when backing is left in place, it obscures effective detection of significant flaws that may exist at the weld root. These flaws represent a significantly more severe notch condition than does the backing itself.

In new construction, as stated in FEMA-353, Recommended Specifications and Quality Assurance Guidelines for Steel Moment-Frame Construction for Seismic Applications, or in modification of existing joints conducted as part of an upgrade project, it is recommended that backing be removed from beam bottom flange joints, to allow identification and correction of weld root flaws. This is not recommended for top flange joints because the stress condition at the top flange is less critical and less likely to result in initiation of fracture, even if some weld root flaws are present. Also, as a result of position, it is far less likely that significant flaws will be incorporated in top flange joints.

Weld tabs represent another source of discontinuity at the critical weld location. Additionally, the weld within the weld tab length is likely to be of lower *quality and more prone to flaws than the body of the weld. Flaws in the weld tab area can create stress concentrations and crack starters and for this reason their removal is recommended. It is important that the process of removal of the runoff tabs not be, of itself, a cause of further stress concentrations, and therefore, FEMA-353 recommends that the workmanship result in smooth surfaces, free of defects.*

Removal of existing backing and weld tabs as a sole means of building upgrade is not recommended. Laboratory testing demonstrates that existing unreinforced welded type FR connections made with low notch toughness weld metal are incapable of ductile performance, even with the removal of these stress rising features. However, they should be removed as part of any program of more substantial upgrades of connections.

6.4.2.7 Reinforcing Fillet Welds and Weld Overlays

When weld backing is removed, the weld should be reinforced with a fillet weld. The size of the weld should be sufficient to cover the root of the existing Complete Joint Penetration weld, and not less than ¼ -in. The profile of the fillet should be as described in Section 5.4 of *AWS D1.1* with a transition free from undercut, except as permitted by *AWS D1.1*.

One method for improving the performance of existing unreinforced connections with low notch toughness weld metal is to reinforce the existing welded joints with weld overlays. This method, which is described in *FEMA-352 Recommended Postearthquake Evaluation and Repair Criteria for Welded Steel Moment-Frame Buildings,* is not prequalified for any specific performance capability, though it is known to be capable of some significant performance improvement.

Commentary: Limited testing on the use of built-up welds (overlay welds) as a means of repairing and reinforcing welded connections of smaller-sized beams in existing buildings has been performed. This upgrade technique has not been prequalified with regard to performance capability as insufficient laboratory test data are available at this time to qualify its use and provide the necessary statistical data on its performance.

6.4.2.8 Weld Access Hole Size, Shape, Workmanship

New welded moment-resisting connections should utilize weld-access hole configurations as shown in Figure 6-8, except as otherwise noted in specific details in these *Recommended Criteria*. Criteria for cutting and finishing of weld access holes are provided in *FEMA-353, Recommended Specifications and Quality Assurance Guidelines for Steel Moment-Frame Construction for Seismic Applications.*

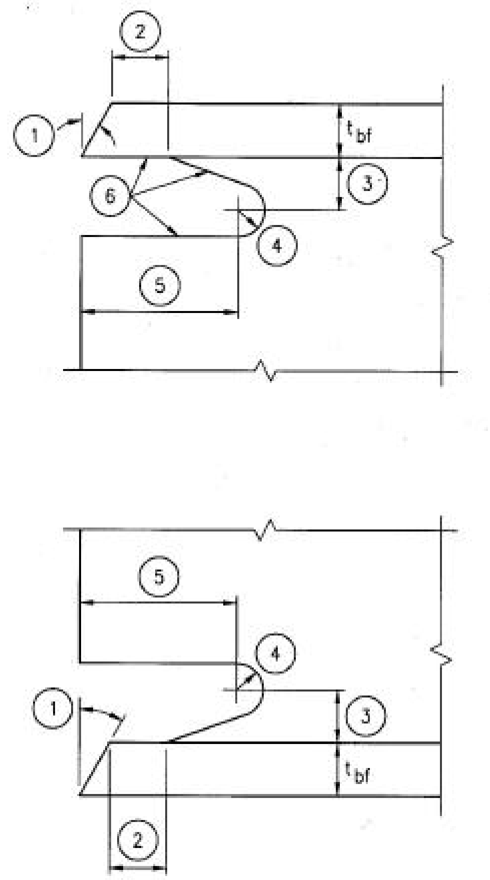
Commentary: The size, shape, and workmanship of weld-access holes can affect connection strength in several different ways. If the hole is not large enough, this restricts welder access to the joint and increases the probability of low quality *joints. Depending on the size and shape of the weld access hole plastic strain demands in the welded joint and in the beam flange at the toe of the weld access hole can be significantly affected. Laboratory tests of unreinforced connections fabricated with tough weld filler metals have indicated that these connections frequently fail as a result of low cycle fatigue of the beam flange material at the toe of the weld access hole, as a result of the strain concentrations introduced by this feature. The configuration shown in Figure 6-8 was developed as part of the program of research conducted under this project and appears to provide a good balance between adequate welder access and minimization of stress and strain concentration. For further discussion of weld access holes, see FEMA-355D, State of the Art Report on Connection Performance.*

6.4.2.9 Welding Quality Control and Quality Assurance

FEMA-353, *Recommended Specifications and Quality Assurance Guidelines for Steel Moment-Frame Construction for Seismic Applications,* contains recommendations for quality control and quality assurance for steel moment frames and connections intended for seismic applications. Recommended inspections are divided into two categories: Process and Visual Inspection, and Nondestructive Testing. For each category, different levels of inspection are specified depending on the anticipated severity of loading, or demand (Seismic Weld Demand Category) and the consequences of welded joint failure (Seismic Weld Consequence Category). All welded joints in the Seismic Force Resisting System should be categorized according to the applicable Consequence and Demand Categories, using the following form: “QC/QA Category BH/T”, where the first letter (in this case B) indicates the Demand Category, the second letter (in this case H) indicates the Consequence Category and the third letter, either T or L indicates that primary loading is either transverse or longitudinal, respectively. The various categories are described in detail in the referenced document. For the prequalified connection upgrades described in these *Recommended Criteria*, the appropriate categories have been preselected and are designated in information accompanying the prequalification.

Commentary: FEMA-353 describes the Demand(A,B,C) and Consequence (H,M,L) Categories and indicates the appropriate levels of Visual and nondestructive testing (NDT) inspection for each combination of demand and consequence. The degree of inspection recommended is highest for the combination of high demand (Category A) with high consequence (Category H) and, conversely, less inspection is required for low demand (Category C) with low consequence (Category L). Intermediate degrees of inspection apply for intermediate categories.

Tolerances shall not accumulate to the extent that the angle of the access hole cut to the flange surface exceeds 25°.



Notes:
1. Bevel for groove weld selected.
2. Larger of t_{bf} or ½ inch (plus ½ t_{bf}, or minus ¼ t_{bf}).
3. ¾ t_{bf} to t_{bf} . ¾" min (± ¼ inch).
4. 3/8" min. radius (plus not limited, or minus 0)
5. 3 t_{bf} (± ½ inch).
6. See *FEMA-353, Recommended Specifications and Quality Assurance Guidelines for Steel Moment-Frame Construction for Seismic Applications,* for fabrication details including cutting methods and smoothness requirements.

Figure 6-8 Recommended Weld Access Hole Detail

6.4.3 Other Design Issues for Welded Connections

6.4.3.1 Continuity Plates

Unless project-specific connection qualification testing is performed to demonstrate that beam flange continuity plates are not required, moment-resisting connections should be provided with beam flange continuity plates across the column web when the thickness of the column flange is less than the value given either by Equation 6-8 or 6-9:

$$t_{cf} < 0.4\sqrt{(1.8 b_f\, t_f\, F_{yb} / F_{yc})} \tag{6-8}$$

$$t_{cf} < b_f / 6 \tag{6-9}$$

where:

t_{cf} = minimum required thickness of column flange when no continuity plates are provided, inches
b_f = beam flange width, inches
t_f = beam flange thickness, inches
F_{yb} = minimum specified yield stress of the beam flange, ksi
F_{yc} = minimum specified yield stress of the column flange, ksi

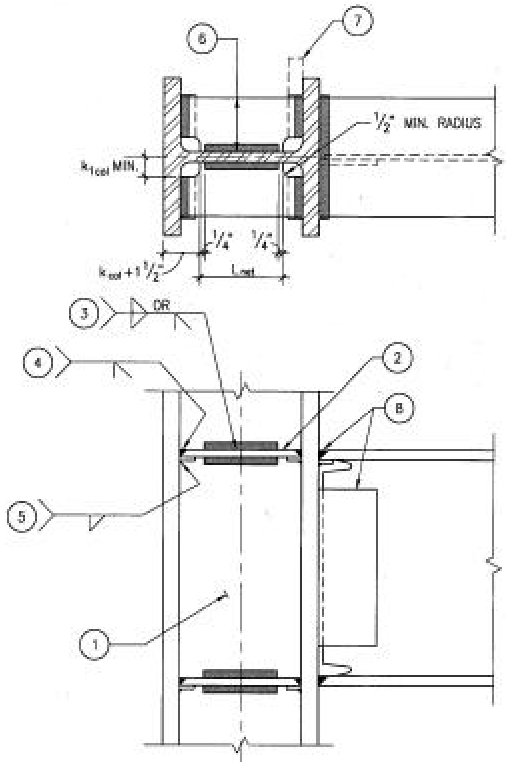
Where continuity plates are required, the thickness of the plates should be determined according to the following:

- For one-sided (exterior) connections, continuity plate thickness should be at least one-half of the thickness of the beam flanges.
- For two-sided (interior) connections, the continuity plates should be equal in thickness to the thicker of the two beam flanges entering the connection on either side of the column.
- The plates should also conform to Section K1.9 of *AISC-LRFD Specifications*.

Continuity plates should be welded to column flanges using complete joint penetration (CJP) welds as shown in Figure 6-9. Continuity plates should be welded to the web, as required, to transmit the shear forces corresponding to development of the axial strength — of the CJP weld at one end of the connection, for one-sided connections, and that at both ends, for two-sided connections.

Commentary: Following the 1994 Northridge earthquake, some engineers postulated that the lack of continuity plates was a significant contributing factor to the failure of some connections. This was partially confirmed by initial tests conducted in 1994 in which several specimens without continuity plates failed while some connections with these plates successfully developed significant ductility. Based on this, FEMA-267 recommended that all connections be provided with continuity plates. The AISC Seismic Provisions (AISC, 1997), which was published after FEMA-267, relaxed this criteria and states that continuity plates should be provided to match those in connections tested to obtain qualification.

Research conducted by this project tends to confirm that where the flange thickness of columns is sufficiently thick, continuity plates may not be necessary. Equation 6-8 was the formula used by AISC to evaluate column flange continuity plate requirements prior to the 1994 Northridge earthquake. It appears that this formula is adequate to control excessive column flange prying provided that the beam flanges are not too wide. Studies reported in FEMA-355D suggest that the ratio of beam flange width to column flange thickness is also important. Tests with a ratio of 5.3 (W36x150 beam with W14x311 column) showed little difference in performance with or without continuity plates, while tests with a ratio of 6.8 (W36x150 beam with W27x258 column) showed some difference of performance. The factor of 6 in Equation 6-9 was selected by judgment based on these tests.



Notes

1. Web doubler plate where required by Section 6.4.3.2. See the *AISC Seismic Provisions* Section 9.3c, Commentary C9.3, and Figures C-9.2 and C-9.3 for options and connection requirements. QC/QA Category *BL/L* requirement for all welds.
2. Continuity plate as required per 6.4.3.1.
3. Required total weld strength = $0.6t_{pl}\left(L_{net}\right)F_{y\,pl}$. QC/QA Category *BL/L*.
4. CJP typical. QC/QA Category *BM/T*.
5. AISC minimum continuous fillet weld under backing.
6. Minimum width to match beam flange. Preferred alternative: extend plate flush with column flanges.
7. Remove weld tabs to ¼" maximum from edge of continuity plate. Grind end of weld smooth (250 μ-in), not flush. Do not gouge column flange.
8. Beam connection, see individual prequalifications.

Figure 6-9 Typical Continuity and Doubler Plates

6.4.3.2 Panel Zone Strength

Moment-resisting connections should be proportioned either so that shear yielding of the panel zone initiates at the same time as flexural yielding of the beam elements, or so that all yielding occurs in the beam. The following procedure is recommended:

Step 1: Calculate *t*, the thickness of the panel zone that results in simultaneous yielding of the panel zone and beam from the following relationship:

$$t = \frac{C_y M_c \frac{h - d_b}{h}}{(0.9)\ 0.55 F_{yc} R_{yc} d_c \left(d_b - t_{fb}\right)} \tag{6-10}$$

where:

h is the average story height of the column, measured from the midpoint of the column above the beam to the midpoint of the column below the beam.

R_{yc} is the ratio of the expected yield strength of the column material to the minimum specified yield strength, in accordance with the 1997 *AISC Seismic Provisions*.

M_c and C_y are the coefficients defined in Section 6.3.7 and Section 6.3.8 of these *Recommended Criteria*, respectively, and other terms are as defined in the *AISC-LRFD Specifications.*

Step 2: If *t*, as calculated, is greater than the thickness of the column web, provide doubler plates, or increase the column size to a section with adequate web thickness.

Where doubler plates are required, the thickness should be determined as described above, and they should be proportioned and welded as described in the 1997 *AISC Seismic Provisions*. QC/QA Category BL/L procedures are defined in *FEMA-353*.

For connections designed using project-specific qualifications, the panel zone strength should match that of the tested connections.

Commentary: Several aspects of the methodology for the design of panel zones, as contained in the 1997 AISC Seismic Provisions, are considered to require revision, based on studies conducted by this project. As described in FEMA-355D, the best performance is likely to be achieved when there is a balance of beam bending and panel zone distortion. The equations given are intended to provide panel zones that are just at the onset of yielding at the time the beam flange begins to yield.

The procedure recommended in this design criteria varies significantly from that contained in the 1997 AISC Seismic Provisions, but the results are not dramatically different. For most column sizes results will be similar to methods used in the past. For columns with thick flanges, the methods herein will result in the need for moderately thicker panel zones than in the past.

6.4.3.3 Connections to Column Minor Axis

Connections to the minor axis of a column should be qualified by testing following the procedures of Section 6.9. If minor-axis connections are to be used in conjunction with major-axis connections to the same column, the testing program should include biaxial bending effects at the connection.

Commentary: In general, the prequalified connections have not been tested for use with columns oriented so that beams connect to the minor axis of the column. Two tests of Reduced Beam Section connections in this orientation were conducted, and indicated good performance. These tests were conducted to provide a general indication of the possible performance of weak axis connections, but are not considered to comprise a sufficient database for prequalification of such connections.

6.4.3.4 Attachment of Other Construction

Welded or bolted attachment for exterior facades, partitions, ductwork, piping, or other construction should not be placed in the hinging area of moment frame beams. The hinging area is defined as one half of the beam depth on either side of the theoretical hinge point as described in the prequalification data table for each connection detail. It is recommended that bolt holes for this type of construction not be permitted between the face of the column and six inches, minimum, beyond the extreme end of the hinging area. Outside the described area, a calculation should be made to ensure sufficient net section to avoid fracture, based on moments calculated using the expected moment at the hinge point. Welding between the column face and the near edge of the hinging area should be carefully controlled to avoid creation of stress concentrations and application of excessive heat. Specifications and drawings should clearly indicate that anchorage shall not be made in the areas described and this should be coordinated with the architect and other members of the design team.

Commentary: It is common for precast panels and other facade elements, as well as other construction, to be anchored to members of the steel frame through the use of welds, bolts, powder-driven fasteners, or other fasteners. Such anchorage is often not considered by the engineer and is not performed with the same care and quality control as afforded the main building structure. Such anchorage, when made in an area of high stress, can lead to stress concentrations and potential fracture.

6.4.4 Bolted Joint Requirements

6.4.4.1 Existing Conditions

When evaluating existing structures, the condition of bolted connections should be determined based on the AISC and Research Council on Structural Connections (RCSC) specifications appropriate to the design and construction years, and on the following criteria:

- Representative samples of bolts should be inspected to determine markings and classifications. Where bolts cannot be properly identified visually, representative samples should be removed and tested to determine tensile strength in accordance with *ASTM F606* and the bolt classified accordingly. Alternatively, bolts may be assumed to be A307.
- Any evidence of yielding in the connection plates indicates that the high-strength bolts are effectively in the snug-tight condition regardless of their original installation condition. If bolts have been identified as ASTM A325 and are not in a snug-tight condition they should

be re-tightened or replaced. If bolts have been identified as ASTM A490 and are not in a snug-tight condition, they should be replaced. Re-tightening or installation of bolts should be to a pretensioned condition in accordance with the 1997 AISC or 1996 RCSC criteria.

6.4.4.2 Connection Upgrades

When upgrading existing connections, the capacity of bolted elements of the connection shall be determined based on the AISC and RCSC specifications appropriate to the design and construction years, and the following criteria:

- Bolts intended to transfer load in the shear/bearing mode should be installed according to the slip critical criteria.
- Bolts intended to transfer load by tension should be pre-tensioned.
- Bolts intended for use in proprietary connections, such as a viscous damping system, should be installed using the instructions applicable to the test data for the system.
- Bolted joints should not be upgraded by sharing loads with weld reinforcement. Any welded reinforcement shall be designed to transfer all the load, independent of the bolt capacity.

6.5 Prequalified Connection Details – General

Prequalified connection and connection upgrade details are permitted to be used for moment frame connections for the types of moment frames and ranges of the various design parameters indicated in each prequalification description. Project-specific testing should be performed to demonstrate the adequacy of connection and upgrade details that are not listed herein as prequalified, or are used outside the range of parameters indicated in the prequalification. Designers should follow the procedures outlined in Section 6.9 for use of nonprequalified connection and upgrade details.

Commentary: The following criteria were applied to connection and upgrade details listed as prequalified:

1. *There is sufficient experimental and analytical data on the connection performance to establish the likely yield mechanisms and failure modes for the connection.*
2. *Rational models for predicting the resistance associated with each mechanism and failure mode have been developed.*
3. *Given the material properties and geometry of the connection, a rational procedure can be used to estimate which mode and mechanism controls the behavior and the deformation capacity (that is, the drift angle) that can be attained from the controlling conditions.*
4. *Given the models and procedures, the existing data base is adequate to permit assessment of the statistical reliability of the connection.*

Some of the connection and upgrade details in the following sections are only prequalified for use in Ordinary Moment Frames (OMFs), while others are prequalified for both OMF and Special Moment Frame (SMF) use. In general, when a connection is qualified for use in SMF systems, it is also qualified for use in OMF systems, with fewer restrictions on size, span, and other parameters than are applied to the SMF usage. Very little extrapolation has been applied in the prequalification limitations for SMFs, while some judgement has been applied to permit extrapolation for OMFs, based on the significantly lower rotational demands applicable to those systems.

6.5.1 Load Combinations and Resistance Factors

Design procedures for prequalified connection upgrades contained in Section 6.6 are formatted on an expected strength basis, as opposed to either a Load and Resistance Factor Design basis or Allowable Stress Design basis. Loading used in these design formulations is generally calculated on the basis of the stresses induced in the assembly at anticipated yielding of the beam-column connection assembly. Where these design procedures require that earthquake loading be applied simultaneously with dead and live loading, the applicable load combinations of the 1997 *AISC Seismic Provisions* apply. Resistance factors should not be applied except as specifically required by the individual design procedure.

6.6 Prequalified Connection Upgrades

This section provides prequalification data for various alternative types of welded steel moment-frame (WSMF) connection upgrade details. Table 6-6 lists the various alternative connection upgrade details that have been prequalified, together with the structural system (SMF or OMF) for which they are prequalified for use in Simplified Upgrade, and reference to the section of these *Recommended Criteria* where detailed information may be found. Refer to these individual reference sections for specific limits on the applicability of the prequalification, for specific performance data for use with Systematic Upgrade and for specific design procedures and details.

Table 6-6 Prequalified Welded Fully Restrained Connection Upgrade Details

Connection Type		Criteria Section	Structural System
Improved welded unreinforced flange	IWURF	6.6.1	OMF
Welded bottom haunch	WBH	6.6.2	OMF, SMF
Welded top and bottom haunch	WTBH	6.6.3	OMF, SMF
Welded cover plated flange	WCPF	6.6.4	OMF, SMF

Commentary: FEMA-355D – State of the Art Report on Connection Performance, provides extensive information on the testing and performance of these connections that is not repeated in this document. The data presented in FEMA-355D have been used in support of development of the prequalification performance data, design procedures, and limitations on design parameters for these connections presented herein.

6.6.1 Improved Welded Unreinforced Flange (IWURF) Connection

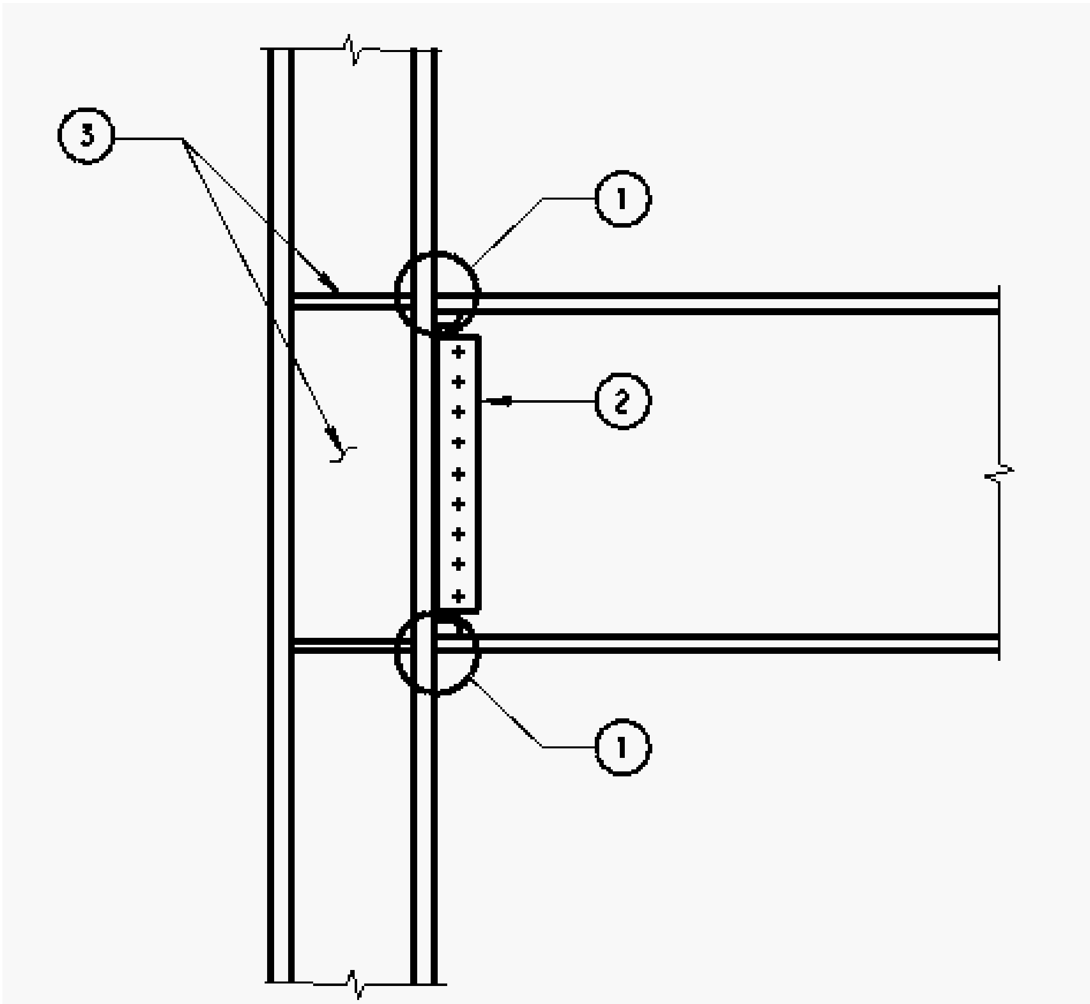
This section provides recommended criteria for design of connection upgrades intended to improve existing unreinforced, welded flange connections by improving the existing welded joints in the connection. This connection upgrade is prequalified only for Ordinary Moment Frame applications. Upgrade is accomplished through replacement of existing complete joint penetration groove welds of low-notch-toughness material and potentially having significant root defects, with new welds conforming to current construction requirements for welded steel moment-frame construction as shown in Figures 6-10 and 6-11. In addition, other elements of the connection, including panel zones and column flanges are reinforced, as required, to conform to the general recommendations of Section 6.4. Table 6-7 tabulates the limits of applicability of this prequalified connection upgrade and associated performance qualification data.

Commentary: This connection upgrades the typical pre-Northridge "prescriptive connection" commonly in use prior to the 1994 Northridge earthquake. After significant study, it has been concluded that with several improvements this connection can be made to perform reliably in frames designed as Ordinary Moment Frames as long as beam sizes are limited as indicated in Table 6-7.

The improvements required for this connection include the following:

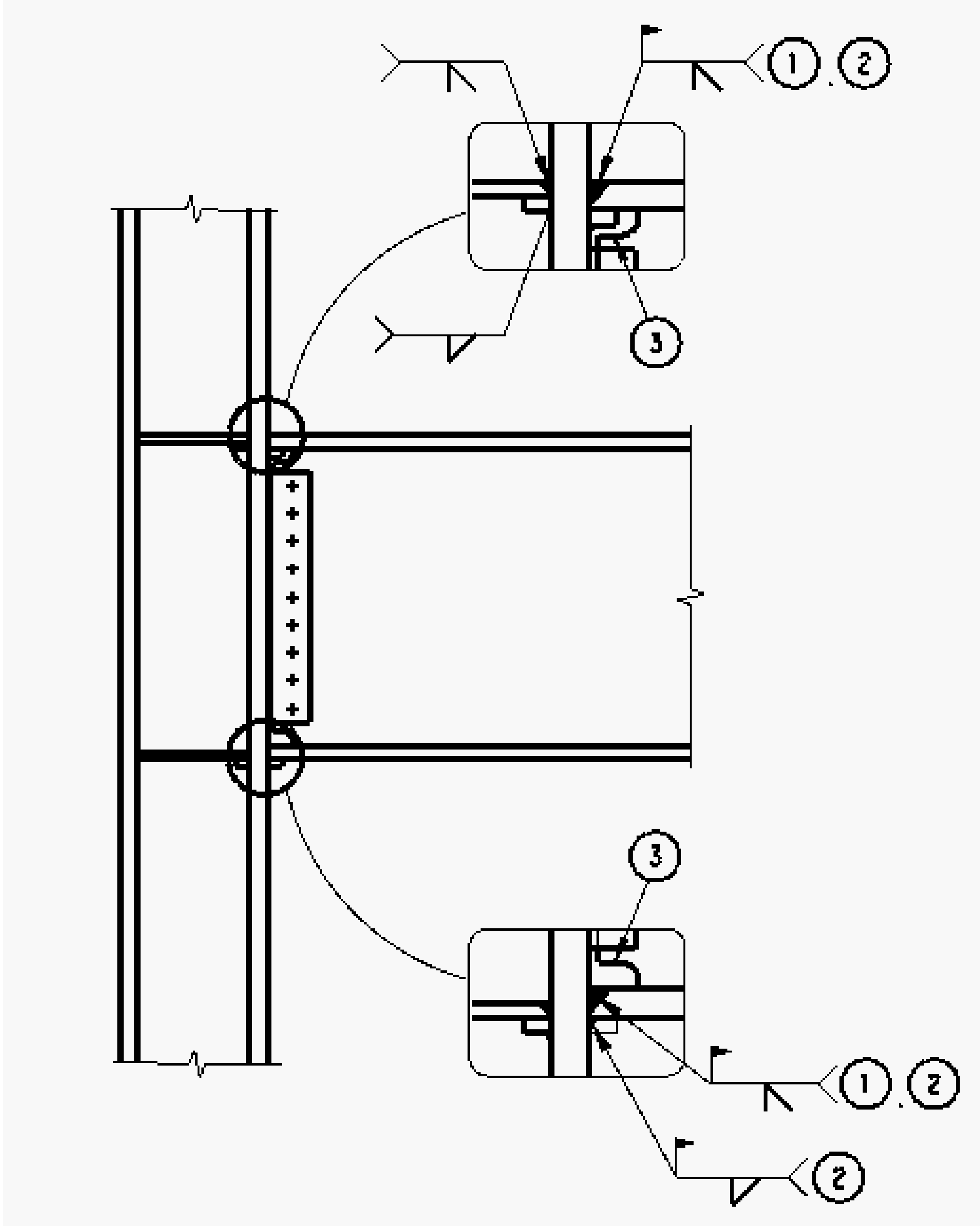
1. *Removal of existing low-toughness weld metal and replacement with weld metal with appropriate toughness;*
2. *Removal of bottom flange weld backing, back-gouging and addition of a reinforcing weld;*
3. *Removal of weld tabs;*
4. *Improvements to weld quality control and quality assurance requirements and methods.*

For best performance of this connection type some limited panel zone yielding is beneficial. For this reason, it is recommended that panel zones not be over-reinforced.



Notes:
1. See Figure 6-11 for welding requirements at these locations.
2. Existing bolted shear tab.
3. Existing or added continuity plates and web doubler plate. See Figure 6-9.

Figure 6-10 Improved Welded Unreinforced Flange Connection



Notes:
1. Gouge out existing weld at both the top and bottom flange and prepare joints for new weld.
2. Complete joint penetration groove weld at top and bottom flanges. At top flange, either (A), remove weld backing, backgouge, and add 5/16" minimum fillet weld, or (B), leave backing in place and add 5/16" fillet under backing. At bottom flange, remove weld backing, backgouge, and add 5/16" minimum fillet weld. Weld is QC/QA Category AH/T.
3. Existing weld access hole to remain unmodified.

Figure 6-11 Welding Requirements at Improved Welded Unreinforced Flange Connection

Table 6-7 Prequalification Data for Improved Welded Unreinforced Flange Connections

Applicability Limits	
General:	
Applicable systems	OMF
Hinge location distance s_h	d_c / 2 + d_b / 2
Critical Beam Parameters:	
Depth	W36 and shallower
Minimum span-to-depth ratio	7
Flange thickness	1” maximum
Permissible material specifications	A7, A36, A572 Gr. 50
Critical Column Parameters:	
Depth	Not limited
Permissible material specifications	A7, A36, A572 Gr. 50
Beam/Column Relations:	
Panel zone strength	Section 6.4.3.2, C_{pr} = 1.1
Column/beam bending strength	No requirement (OMF)
Connection Details:	
Web connection	Existing bolted shear tab
Continuity plate thickness	Section 6.4.3.1
Flange welds	Figures 6-10 and 6-11
Weld electrodes	Sections 6.4.2.4 and 6.4.2.5
Weld access holes	Existing weld access hole
Performance Data:	
Strength degradation rotation - $q_{SD,}$ radians	0.031 - 0.0003d_b
Immediate Occupancy rotation - q_{IO}, radians	0.015, but not greater than q_{SD}
Resistance factor, Immediate Occupancy, f	0.9
Collapse Prevention drift angle - q_U, radians	0.060 - 0.0006d_b
Resistance factor, Collapse Prevention, f	0.9

Notes: d_b= beam depth, inches; d_c = column depth, inches.

6.6.1.1 Design Procedure

Step 1: Calculate M_{pr}, at hinge location, s_h, according to methods of Section 6.3.5.

Step 2: Calculate V_p, at hinge location, s_h, according to methods of Section 6.3.6.

Step 3: Calculate M_c, M_f, and C_y as described in Section 6.3.7 and 6.3.8.

Step 4: Calculate the required panel zone thickness using the procedures of Section 6.4.3.2.

Step 5: Check requirements for Continuity Plates according to Section 6.4.3.1.

Step 6: Detail the connection as shown in Figure 6-10 and 6-11.

Commentary: There is more research information available on unreinforced beam-to-column connections than there is on any other type of steel moment-frame connection. Not only were these connections extensively studied prior to the 1994 Northridge earthquake, they have been even more extensively studied in the aftermath. Many of the studies focused on the connection as used in pre-1994 practice, with bolted web connection, and flange welds with unrated or low notch toughness and with backing left in place, while other studies have been focused on improvements to the connection, including those improvements recommended in this section.

These tests give widely scattered results, but in general, indicate that when weld metal with sufficient notch toughness is used and workmanship is maintained at an appropriate level, these connections can reliably perform adequately for service in Ordinary Moment Frame, if not Special Moment Frame systems. Additional information may be found in FEMA-355D, State of the Art Report on Connection Performance.

6.6.2 Welded Bottom Haunch (WBH) Connection

This connection upgrade is accomplished by converting the existing welded unreinforced (WUF) connection into a haunched connection, with a single haunch present at the bottom beam flange. This connection upgrade is prequalified for both OMF and SMF applications. If the weld of the top beam flange to the column is made with weld metal with low or unclassified notch toughness, then, in addition to welding the new haunch at the bottom beam flange, this top beam flange weld must be gouged out and replaced with weld metal conforming to the recommendations of Sections 6.4.2.3 and 6.4.2.4 to obtain SMF service. The general requirements of Section 6.4 should be complied with. Figure 6-12 provides a typical detail for this connection. Table 6-8 presents performance qualification data for the connection. Refer to *AISC Steel Design Guide Series 12* (Gross et al., 1999) for supplemental information to the design procedure given in Section 6.6.2.1.

6.6.2.1 Design Procedure

Step 1: Calculate M_{pr}, at hinge location, s_h, according to methods of Section 6.3.5.

Step 2: Calculate V_p, at hinge location, s_h, according to methods of Section 6.3.6.

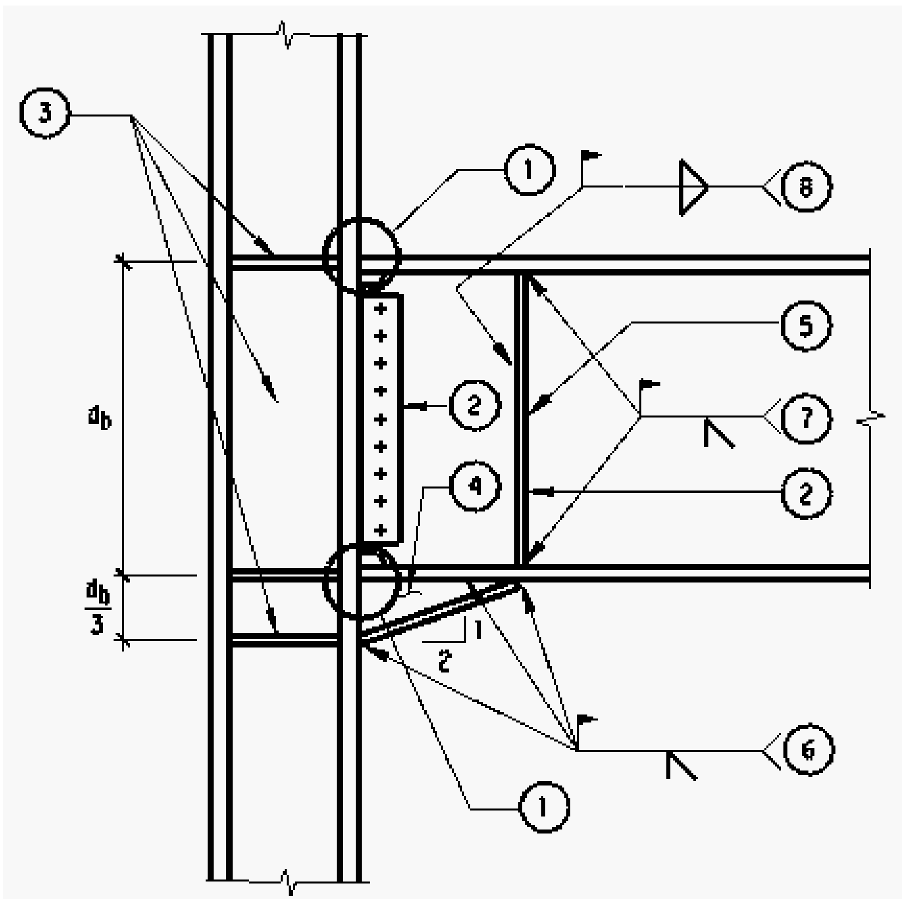
Step 3: Calculate M_c, M_f, and C_y as described in Section 6.3.7 and 6.3.8.

Step 4: Calculate the required panel zone thickness using the procedures of Section 6.4.3.2.

Step 5: Check requirements for Continuity Plates according to Section 6.4.3.1.

Step 6: Size the haunch according to the criteria outlined in AISC Steel Design Guide Series 12.

Step 7: Detail the connection as shown in Figure 6-12.



Notes

1. For OMF connection, existing weld can remain. For SMF connection, see Figure 6-11.
2. Existing bolted shear tab.
3. Existing continuity plates and web doubler plate. See Figure 6-9.
4. WT haunch.
5. New ½”-minimum stiffener plates each side.
6. Haunch welds, see Sections 6.4.2.3 and 6.4.2.4, QC/QA category AH/T.
7. Stiffener CJP welds; see Sections 6.4.2.3 and 6.4.2.4, QC/QA Category BM/T.
8. Stiffener fillet welds, 5/16” minimum. QC/QA Category CL/L.

Figure 6-12 Welded Bottom Haunch (WBH) Connection

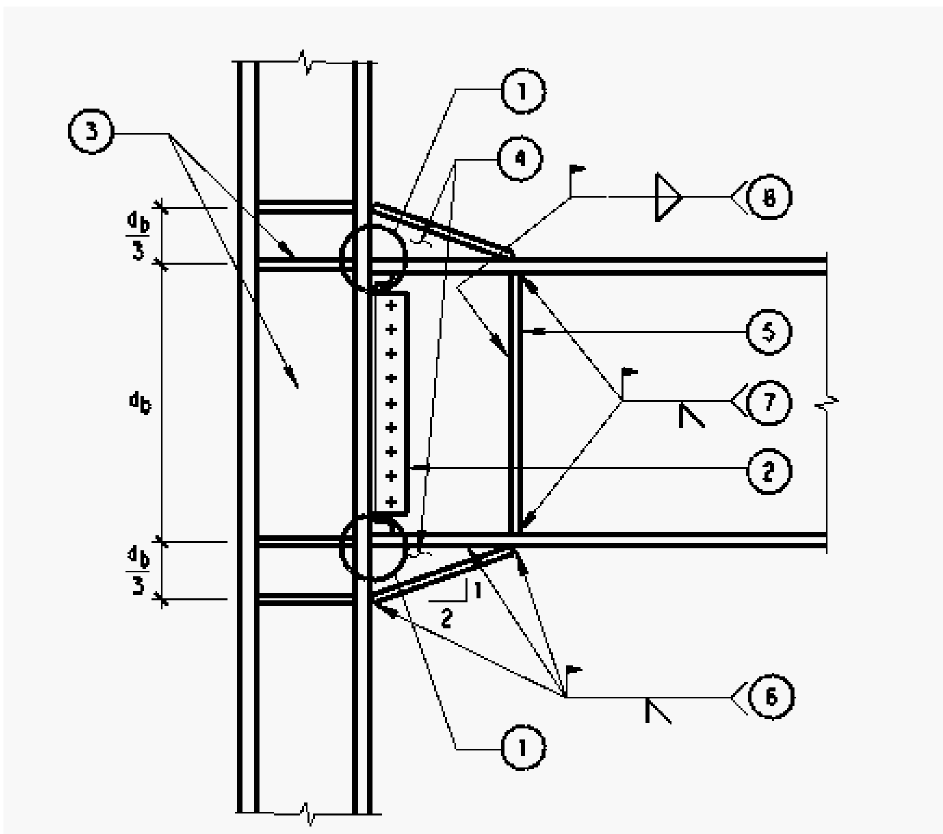
Table 6-8 Prequalification Data for Welded Bottom Haunch (WBH) Connection

Applicability Limits	
General:	
Applicable systems	OMF, SMF
Hinge location distance s_h	$d_c/2 + l_h$ from center of column
Critical Beam Parameters:	
Depth range	Up to W36
Minimum span-to-depth ratio	OMF: 5 SMF: 7
Flange thickness	OMF: 1-1/2" maximum SMF: 1" maximum
Permissible material specifications	A7, A36, A572 Gr. 50
Beam flange welds	OMF: Existing welds can remain. SMF: Sections 6.4.2.3 and 6.4.2.4
Critical Column Parameters:	
Depth	OMF: Not limited SMF: W12, W14
Permissible material specifications	A7, A36, A572 Gr. 50
Beam / Column Relations:	
Panel zone strength	OMF: Section 6.4.3.2, C_{pr} = 1.1 SMF: Section 6.4.3.2
Column/beam bending strength ratio	OMF: No requirement SMF: Section 6.4.1.1
Connection Details:	
Web connection	Existing bolted shear tab
Continuity plate thickness	At beam flanges: Section 6.4.3.1 At haunch: match haunch width and thickness
Haunch welds	Sections 6.4.2.3 and 6.4.2.4
Details of Haunch Design:	
Haunch size and strength criteria	Haunch to be sized by criteria as outlined in *AISC Steel Design Guide Series 12* (Gross et al., 1999)
Performance Data:	
Strength degradation rotation - q_{SD}, radians	0.038
Immediate Occupancy rotation - q_{IO}, radians	0.020
Resistance factor, Immediate Occupancy, f	0.9
Collapse Prevention drift angle - q_U – radians	0.06
Resistance factor, Collapse Prevention, f	0.9

Note: d_c = column depth

6.6.3 Welded Top and Bottom Haunch (WTBH) Connection

This connection upgrade is accomplished by attaching a new welded haunch to both the top and bottom flanges of the existing beam connection. This connection upgrade is prequalified for both OMF and SMF applications. Existing welds in the connection need not be gouged out, nor replaced, for OMF applications. For SMF applications, in addition to installing the new haunches, if the beam flange welds to the column are made with weld metal of unclassified or low notch toughness, these welds must be gouged out and replaced with weld metal conforming to the recommendations of Sections 6.4.2.3 and 6.4.2.4. Design is accomplished to accommodate the general requirements of Section 6.4. Figure 6-13 shows a typical detail for this connection. Table 6-9 provides performance qualification data.



Notes
1. For OMF connection, weld can remain. For SMF connection, see Figure 6-11.
2. Existing bolted shear tab.
3. Existing continuity plates and web doubler plate. See Figure 6-9.
4. WT haunches.
5. New ½"-minimum stiffener plate each side.
6. Haunch welds, see Sections 6.4.2.3 and 6.4.2.4, QC/QA category AH/T.
7. Stiffener CJP welds; see Sections 6.4.2.3 and 6.4.2.4, QC/QA Category BM/T.
8. Stiffener fillet welds, 5/16” minimum. QC/QA Category CL/L.

Figure 6-13 Welded Top and Bottom Haunch (WTBH) Connection

Table 6-9 Prequalification Data for Welded Top and Bottom Haunch (WTBH) Connections

Applicability Limits	
General:	
Applicable systems	OMF, SMF
Hinge location distance s_h	d_c /2 + l_h from center of column
Critical Beam Parameters:	
Depth range	Up to W36
Minimum span-to-depth ratio	OMF: 5 SMF: 7
Flange thickness	OMF: 1-1/2” maximum SMF: 1” maximum
Permissible material specifications	A7, A36, A572 Gr. 50
Beam flange welds	OMF: Existing welds can remain. SMF: Sections 6.4.2.3 and 6.4.2.4
Critical Column Parameters:	
Depth	OMF: Not limited SMF: W12, W14
Permissible material specifications	A7, A36, A572 Gr. 50
Beam / Column Relations:	
Panel zone strength	OMF: Section 6.4.3.2, C_{pr} = 1.1 SMF: Section 6.4.3.2
Column/beam bending strength ratio	OMF: No requirement SMF: Section 6.4.1.1
Connection Details:	
Web connection	Existing bolted shear tab
Continuity plate thickness	At beam flanges: Section 6.4.3.1 At haunch: match haunch width and thickness
Haunch welds	Section 6.4.2.3 and 6.4.2.4
Details of Haunch Design:	
Haunch size and strength criteria	Haunch to be sized by criteria as outlined in *AISC Steel Design Guide Series 12* (Gross et al., 1999)
Performance Data:	
Strength degradation rotation - $q_{SD,}$ radians	0.038
Immediate Occupancy rotation - q_{IO}, radians	0.02
Resistance factor, Immediate Occupancy, f	0.9
Collapse Prevention drift angle - q_U – radians	0.058
Resistance factor, Collapse Prevention, f	0.9

Note: d_c = depth of column, inches

6.6.3.1 Design Procedure

Step 1: Calculate M_{pr}, at hinge location, s_h, according to methods of Section 6.3.5.

Step 2: Calculate V_p, at hinge location, s_h, according to methods of Section 6.3.6.

Step 3: Calculate M_c, M_f, and C_y as described in Section 6.3.7 and 6.3.8.

Step 4: Calculate the required panel zone thickness using the procedures of Section 6.4.3.2.

Step 5: Check requirements for Continuity Plates according to Section 6.4.3.1.

Step 6: Size the haunches according to the criteria outlined in *AISC Steel Design Guide Series 12* (Gross, et al., 1999).

Step 7: Detail the connection as shown in Figure 6-13.

6.6.4 Welded Cover Plated Flange (WCPF) Connection

This connection upgrade is accomplished by attaching new cover plates to both the top and bottom flanges of the existing beam. This connection upgrade is prequalified for both OMF and SMF applications. Existing welds in the connection need not be gouged out, nor replaced, for OMF applications. In addition to welding the new cover plates, if the beam flange welds to the column are made with welds having notch toughness that is either not classified or low, this weld must be gouged out and replaced with weld metal conforming to the recommendations of Sections 6.4.2.3 and 6.4.2.4 to obtain SMF service. Design is accomplished to accommodate the general requirements of Section 6.4. Figure 6-14 shows a typical detail for this connection. Table 6-10 provides prequalification limitations.

6.6.4.1 Design Procedure

Step 1: Calculate M_{pr}, at hinge location, s_h, according to methods of Section 6.3.5.

Step 2: Calculate V_p, at hinge location, s_h, according to methods of Section 6.3.6.

Step 3: Calculate M_c, M_f, and C_y as described in Section 6.3.7 and 6.3.8.

Step 4: Calculate the required panel zone thickness using the procedures of Section 6.4.3.2.

Step 5: Check requirements for Continuity Plates according to Section 6.4.3.1.

Step 6: Size the cover plates. When cover plates are to be field welded, the top cover plate should be narrower than the beam flange and the bottom cover plate should be wider. The area of the cover plates should be sized to satisfy the following relationship:

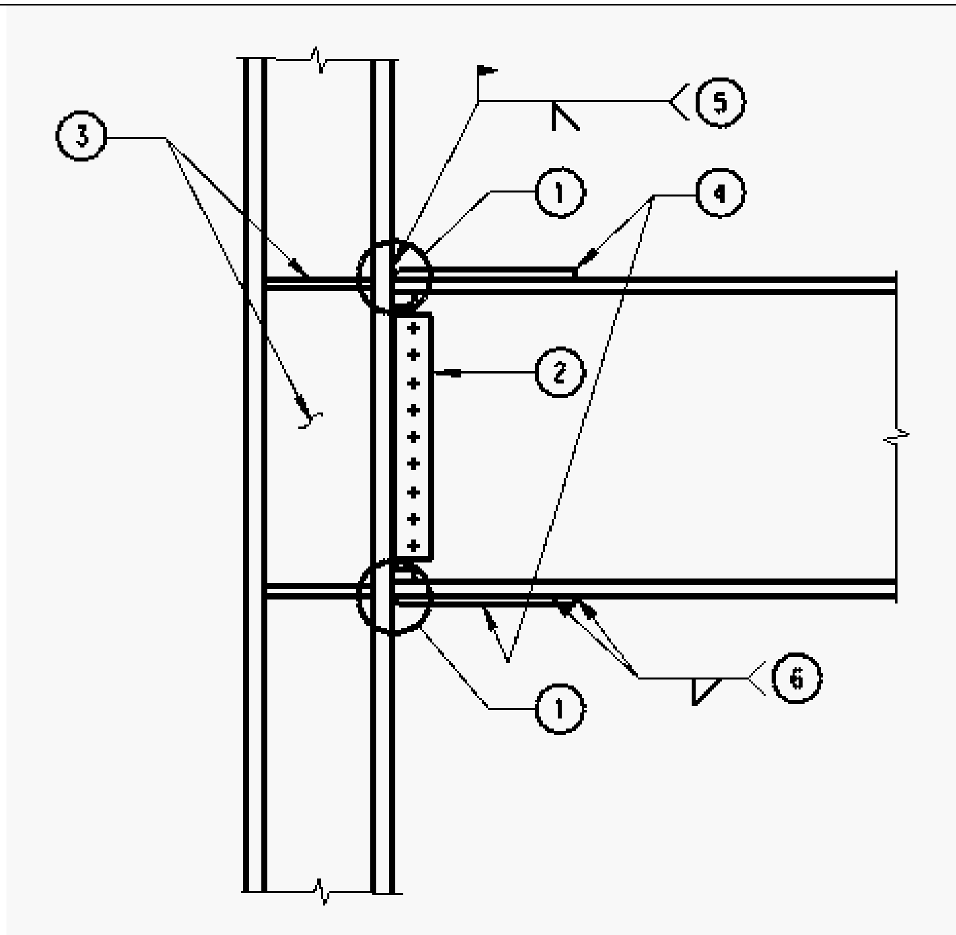
$$\left(kZ_b + A_{cp}\left(d_b + t_{cp}\right)\right)F_y \geq M_f \tag{6-11}$$

where:

k = 0.4 for OMF and 1.0 for SMF connections
A_{cp} = cross-section area of the cover plate, square inches
d_b = depth of the beam, inches
t_{cp} = thickness of the cover plate, inches

The remainder of the terms are as defined in Section 6.3 and 6.4.

Step 7: Detail the connection as shown in Figure 6-14.



Notes:
1. For OMF connection, weld can remain. For SMF connection, see Figure 6-11.
2. Existing bolted shear tab.
3. Existing continuity plates and web doubler plate. See Figure 6-8.
4. Cover plates.
5. Cover plate CJP welds, see Section 6.4.2.3 and 6.4.2.4, QC/QA Category AH/T.
6. Cover plate fillet welds, QC/QA Category BH/L.

Figure 6-14 Welded Cover Plated Flange (WCPF) Connection

Table 6-10 Prequalification Data for Welded Cover Plated Flange Connections

Applicability Limits	
General:	
Applicable systems	OMF, SMF
Hinge location distance s_h	d_c /2 + l_{cp} from center of column
Critical Beam Parameters:	
Depth range	Up to W36
Minimum span-to-depth ratio	OMF: 5 SMF: 7
Flange thickness	OMF: 1-1/2: maximum SMF: 1” maximum
Permissible material specifications	A7, A36, A572 Gr. 50
Beam flange welds	OMF: Existing welds can remain. SMF: Sections 6.4.2.3 and 6.4.2.4.
Critical Column Parameters:	
Depth	OMF: Not limited SMF: W12, W14
Permissible material specifications	A7, A36, A572 Gr. 50
Beam / Column Relations:	
Panel zone strength	OMF: Section 6.4.3.2, C_{pr} = 1.1 SMF: Section 6.4.3.2
Column/beam bending strength ratio	OMF: No requirement SMF: Section 6.4.1.1
Connection Details:	
Relative size and proportions of cover plate	Section 6.6.4.1, Step 6.
Web connection	Existing bolted shear tab.
Continuity plate thickness	Section 6.4.3.1
Cover plate welds	Section 6.4.2.3 and 6.4.2.4
Performance Data:	
Strength degradation rotation - $q_{SD,}$ radians	0.066 - 0.0011d_b
Immediate Occupancy rotation - q_{IO}, radians	0.02, but not greater than q_{SD}
Resistance factor, Immediate Occupancy, f	0.9
Collapse Prevention drift angle - q_U, radians	0.066 - 0.0011d_b
Resistance factor, Collapse Prevention, f	0.9

Notes: d_b= beam depth, inches, d_c= column depth

6.7 New Moment Frames and Moment-Resisting Connections

In some cases, it may be desirable to upgrade an existing steel moment-frame building by introducing new steel moment frames. This can be accomplished either with the addition of new framing, or the modification of existing framing not originally intended to participate in lateral resistance. New moment-resisting connections, introduced for such purpose, should be designed in accordance with the design procedures presented in *FEMA-350, Recommended Seismic Design Criteria for New Steel Moment-Frame Buildings,* and constructed in accordance with *FEMA-353, Recommended Specifications and Quality Assurance Guidelines for Steel Moment-Frame Construction for Seismic Applications.* Table 6-11 presents performance data for connections that have been prequalified for use in new construction. The table may be used in assessing the effectiveness of new or modified framing employing these connections to achieve desired performance goals.

Commentary: Upgrade of existing WSMF buildings with the addition of new steel moment frames, or the modification of existing gravity frames to provide lateral resistance, will typically not be an effective upgrade strategy. This is because steel moment frames are inherently flexible and it is unlikely that the addition of new frames, by themselves, will be sufficient to control building drifts to levels that will protect existing WSMF connections from damage.

6.8 Proprietary Connections

This section presents information on several types of fully restrained connection technologies that have been developed on a proprietary basis. These connection technologies are not categorized in these *Recommended Criteria* as prequalified, as the SAC Joint Venture has not examined the available supporting data in sufficient detail to confirm that they meet appropriate prequalification criteria. However, these proprietary connections have been evaluated by some enforcement agencies and found to be acceptable for specific projects and in some cases for general application within the jurisdiction's authority. Use of these technologies without the express permission of the licensor may be a violation of intellectual property rights, under the laws of the United States.

Discussion of several types of proprietary connections are included herein. Other proprietary connections may also exist. Inclusion or exclusion of proprietary connections in these *Recommended Criteria* should not be interpreted as either an approval or disapproval of these systems. The descriptions of these connections contained herein have in each case been prepared by the developer or licensor of the technology. This information has been printed with their permission. Neither the Federal Emergency Management Agency nor the SAC Joint Venture endorses any of the information provided or any of the claims made with regard to the attributes of these technologies or their suitability for application to specific projects. Designers wishing to consider specific proprietary connections for use in their structures should consult both the licensor of the connection and the applicable enforcement agency to determine the applicability and acceptability of the individual connection for the specific design application.

Table 6-11 Performance Data for Prequalified Moment-Resisting Connections for New Framing

	Strength Degradation[1]	Immediate Occupancy		Collapse Prevention[1]	
Connection Type	q_{SD}	q_{IO}[2]	f	q_U	f
Welded Unreinforced Flange, Bolted Web (WUF-B)	$0.031\text{-}0.0003d_b$	0.020	0.9	$0.060\text{-}0.0006d_b$	0.9
Welded Unreinforced Flange, Welded Web (WUF-W)	0.051	0.020	0.9	0.064	0.9
Free Flange (FF)	$0.077\text{-}0.0012d_b$	0.020	0.9	$0.104\text{-}0.0016d_b$	0.9
Reduced Beam Section (RBS)	$0.060\text{-}0.0003d_b$	0.020	0.9	$0.080\text{-}0.0003d_b$	0.9
Welded Flange Plate (WFP)	0.04	0.020	0.9	0.07	0.9
Bolted Unstiffened End Plate (BUEP)	$0.071\text{-}0.0013d_b$	0.020	0.9	$0.081\text{-}0.0013d_b$	0.9
Bolted Stiffened End Plate (BSEP)	$0.071\text{-}0.0013d_b$	0.020	0.9	$0.081\text{-}0.0013d_b$	0.9
Bolted Flange Plate (BFP)	$0.12\text{-}0.0023d_b$	0.020	0.9	$0.10\text{-}0.0011d_b$	0.9
Double Split Tee (DS)	$0.12\text{-}0.0032d_b$	0.020	0.9	$0.14\text{-}0.0032d_b$	0.9

Notes:

Values in this table apply only to connections and framing that comply in all respects with the prequalification limits indicated in *FEMA-350, Recommended Seismic Design Criteria for New Steel Moment-Frame Buildings* and *FEMA-353, Recommended Specifications and Quality Assurance Guidelines for Steel Moment-Frame Construction for Seismic Applications.*

1. For connections that are prequalified in *FEMA-350* for either SMF or OMF service, the values indicated apply for framing and connections that comply with the applicability limits for SMF service. When framing and connections comply with the applicability limits for OMF service but not for SMF service, ½ the tabulated values shall be used.
2. The value of q_{IO} shall not be taken greater than the value for q_{SD}.

6.8.1 Side Plate (SP) Connection

The proprietary Side Plate connection system is a patented technology shown schematically in Figure 6-15 for its application to upgrade of existing construction. Physical separation between the face of the column flange and the end of the beam eliminates peaked triaxial stress concentrations. Physical separation is achieved by means of parallel full-depth side plates that eliminate reliance on through-thickness properties and act as discrete continuity elements to sandwich and connect the beam and the column. The increased stiffness of the side plates inherently stiffens the global frame structure and eliminates reliance on panel zone deformation by providing three panel zones [i.e., the two side plates plus the column's own web]. Top and bottom beam flange cover plates are used, when dimensionally necessary, to bridge the difference between the flange widths of the beam and the column.

This connection system uses all fillet-welded fabrication. All fillet welds are made in either the flat or horizontal position using column tree construction. For new construction, shop fabricated column trees and link beams are erected and joined in the field using one of four link beam splice options to complete the moment-resisting frame. Link beam splice options include a fully welded CJP butt joint, bolted matching end plates, fillet-welded flange plates, and bolted flange plates.

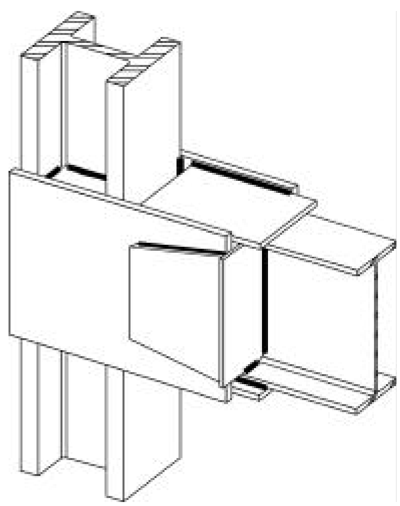


Figure 6-15 Proprietary Side Plate Connection – Application to Existing Construction

All connection fillet welds are loaded principally in shear along their length. Moment transfer from the beam to the side plates, and from the side plates to the column, is accomplished with plates and fillet welds using equivalent force couples. Beam shear transfer from the beam's web to the side plates is achieved with vertical shear plates and fillet welds. The side plates are designed with adequate strength and stiffness to force all significant plastic behavior of the connection system into the beam, in the form of flange and web local buckling centered at a distance of approximately 1/3 the depth of the beam away from the edge of the side plates.

All full-scale cyclic testing of this connection system was conducted at the Charles Lee Powell Structural Research Laboratories, University of California, San Diego, under the direction of Professor Chia-Ming Uang. Testing included both prototype uniaxial and biaxial dual strong axis tests. Independent corroborative nonlinear analyses were conducted by the University of Utah and by Myers, Houghton & Partners, Structural Engineers.

Independent prequalification of this connection system was determined by ICBO Evaluation Service, Inc., in accordance with *ICBO ES Acceptance Criteria for Qualification of Steel Moment-Frame Connection Systems (AC 129-R1-0797),* and was corroborated by the City of Los Angeles Engineering Research Section, Department of Building and Safety. These invoke the qualification procedures contained in *FEMA 267/267A/267B*; *AISC Seismic Provisions for Structural Steel Buildings,* dated April 15, 1997; and *County of Los Angeles Current Position on Design and Construction of Welded Moment-Resisting Frame Systems CP-2,* dated August 14, 1996. Refer to *ICBO Evaluation Service, Inc., Evaluation Report No. 5366,* issued January 1, 1999, and to *City of Los Angeles Research Report: COLA RR 25393* for allowable values and conditions of use. Additional independent jurisdictional scrutiny of this connection system, by Karl H. Frank, Ph.D., Egor P. Popov, Ph.D., C. Mark Saunders, S.E., and Robert L. Schwein, P.E. is contained in the *Los Angeles County Technical Advisory Panel (LACO-TAP) SMRF Bulletin No. 3 on Steel Moment-Resisting Frame Connection Systems, County of Los Angeles, Department of Public Works,* dated March 4, 1997. Additional design information for this connection type may be obtained from the licensor.

The Side Plate connection for upgrade construction differs from its configuration for new construction by featuring an initial opening in each side plate to permit welding access, saving the cut-out pieces of plate for use as closure plates to close the access window after welding is completed. All new welds are fillet welds loaded principally in shear along their length. The existing Complete Joint Penetration (CJP) welds joining the beam flanges to the column flange are removed by airarcing to eliminate reliance on through-thickness properties and triaxial stress concentrations. The existing shear tab of the steel moment-frame beam(s) is left in place to provide gravity support. Existing continuity plates may be left in place to act as horizontal shear plates as depicted in Figure 6-15.

6.8.2 Slotted Web (SW) Connection

This proprietary connection (Seismic Structural Design Associates, Inc. US Patent No. 5,680,738 issued 28 October 1997) is shown schematically in Figure 6-16. It is similar to the popular field welded–field bolted beam-to-column moment frame connection, shown in the current *AISC LRFD* and *ASD* steel design manuals, that has become known as the "pre-Northridge" connection. Based upon surveys of seismic connection damage, modes of fracture, reviews of historic tests, and recent ATC-24 protocol tests, it was concluded by SEAOC (1996 *Blue Book Commentary*) that the pre-Northridge connection is fundamentally flawed and should not be used in the new construction of seismic moment frames. Subsequent finite element analyses and strain gage data from ATC-24 tests of this pre-Northridge connection have shown large stress and strain gradients horizontally across and vertically through the beam flanges and welds at the face of the column. These stress gradients produce a prying moment in the beam flanges at the weld access holes and in the flange welds at the column face that lead to beam flange and weld fractures and column flange divot modes of connection fracture. Moreover, these same studies have also shown that a large component, typically 50%, of the vertical beam shear and all of the beam moment, is carried by the beam flanges/welds in the pre-Northridge connection.

However, by (1) separating the beam flanges from the beam web in the region of the connection and (2) welding the beam web to the column flange, the force, stress and strain distributions in this field welded-field bolted connection are changed dramatically in the following ways:

1. The vertical beam shear in the beam flanges/welds is reduced from typically 50% to typically 3% so that essentially all vertical shear is transferred to the column through the beam web and shear plate.
2. Since most W sections have a flange to beam modulus ratio of $0.65 < Z_{flg} /Z < 0.75$, both the beam web and flange separation and the beam web to column flange weldment force the beam web to resist its portion of the total beam moment.
3. The beam web separation from the beam flange reduces the large stress and strain gradients across and through the beam flanges by permitting the flanges to flex out of plane. Typically, the elastic stress and strain concentration factors (SCFs) are reduced from 4.0 to 5.0 down to 1.2 to 1.4, which dramatically reduces the beam flange prying moment and the accumulated plastic strain and ductility demand under cyclic loading. These attributes enhance and extend the fatigue life of this moment frame connection.
4. The lateral-torsional mode of beam buckling that is characteristic of non-slotted beams is circumvented. The separation of the beam flanges and beam web allow the flanges and web to buckle independently and concurrently, which eliminates the twisting mode of buckling and its associated torsional beam flange/weld stresses. Elimination of this buckling mode is particularly important when the exterior cladding of the building is supported by seismic moment frames that are located on the perimeter of the building.
5. Residual weldment stresses are significantly reduced. The separation of the beam web and flanges in the region of the connection provides a long structural separation between the vertical web and horizontal flange weldments.

The slotted web (SW) connection design rationale that sizes the beam/web separation length, shear plate and connection weldments, is based upon ATC-24 protocol test results and inelastic finite element analyses of the stress and strain distributions and buckling modes. Incorporated in this rationale are the *UBC* and *AISC Load and Resistance Factor Design (LRFD) Specifications* and the *AISC Seismic Design Provisions for Steel Buildings*.

Seismic Structural Design Associates (SSDA) has successfully completed ATC-24 protocol tests on beams ranging from W27x94 to W36x280 using columns ranging from W14x176 to W14x550. None of these assemblies experienced the lateral-torsional mode of buckling that is typical of non-slotted beam and column assemblies.

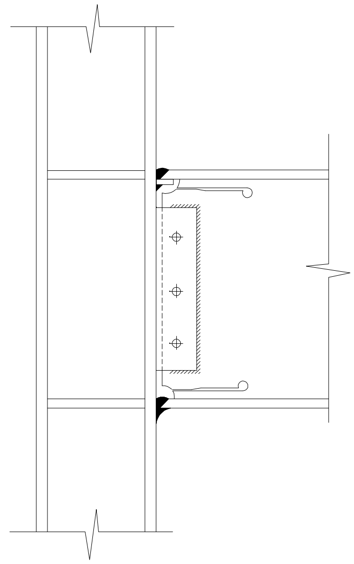


Figure 6-16 Proprietary Slotted Web Connection

Both analytical studies and ATC-24 protocol tests have demonstrated that the Seismic Structural Design Associates (SSDA) Slotted Web connection designs develop the full plastic moment capacity of the beam and do not reduce the elastic stiffness of the beam. All of the above attributes of this proprietary connection enhance its strength and ductility, which makes it applicable for use in retrofit of existing seismic moment frames. Specific qualification and design information for the Slotted Web connection may be obtained from the licensor.

6.8.3 Bolted Bracket (BB) Connection

This connection type is shown schematically in Figure 6-17. Beam shear and flexural stresses are transferred to the column through a pair of heavy bolted brackets, located at the top and bottom beam flanges. The concept of using bolted brackets to connect beams to columns rigidly is within the public domain, but generic prequalification data have not been developed for this connection. One licensor has developed patented steel castings of the bolted brackets, for which specific design qualification data has been prepared. Specific qualification and design information for this connection may be obtained from the licensor.

6.9 Project-Specific Testing of Nonprequalified Connections

This section provides recommended criteria for design and project-specific qualification of connections and connection upgrades for which there is no current prequalification. Recommended criteria are also provided for prequalified details which are to be utilized outside the parametric limitations for a current prequalification. Project-specific qualification includes a program of connection assembly prototype testing, supplemented by a suitable analytical procedure that permits prediction of behavior identified in the testing program.

Commentary: While it is not the intent of these Recommended Criteria to require testing for most situations, there will arise circumstances where proposed *connections do not satisfy prequalification requirements. In these situations, the requirement for testing reflects the view that the behavior of connections under severe cyclic loading cannot be reliably predicted by analytical means alone.*

This suggests that for nonprequalified connections, both laboratory testing and the development of an analytical procedure that predicts the behavior are required. Requiring an analytical procedure, based on testing, develops a design methodology applicable to the design of connections employing slightly different members than actually tested.

Testing is costly and time consuming, and it is the intent of these Recommended Criteria to minimize testing requirements to the extent possible. Test conditions should match the conditions in the structure as closely as possible.

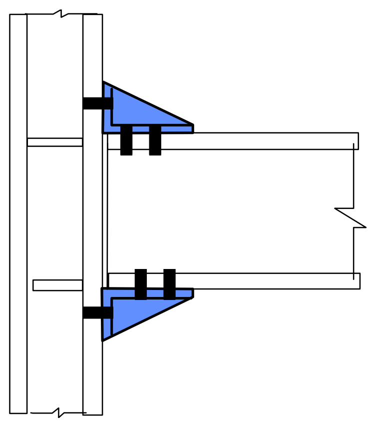


Figure 6-17 Bolted Bracket Connection

6.9.1 Testing Procedure

The testing program should follow the requirements of Appendix S of the 1997 *AISC Seismic Provisions* with the exceptions and modifications discussed below. The program should include tests of at least two specimens for a given combination of beam and column size. The results of the tests should be capable of predicting the median value of the interstory drift angle capacity for the performance states described in Table 6-12. The drift angle capacity q shall be defined as indicated in Figure 6-18. Acceptance criteria should be as indicated in Section 6.9.2.

Table 6-12 Interstory Drift Angle Limits for Various Performance Levels

Performance Level	Symbol	Drift Angle Capacity
Peak Strength	q_{IO}	Taken as that value of q in Figure 6-18 at which peak load resistance occurs.
Strength degradation	q_{SD}	Taken as that value of q in Figure 6-18 at which either failure of the connection occurs or the strength of the connection degrades to less than the nominal plastic capacity, whichever is less
Ultimate	q_U	Taken as that value of q in Figure 6-18 at which connection damage is so severe that continued ability to remain stable under gravity loading is uncertain.

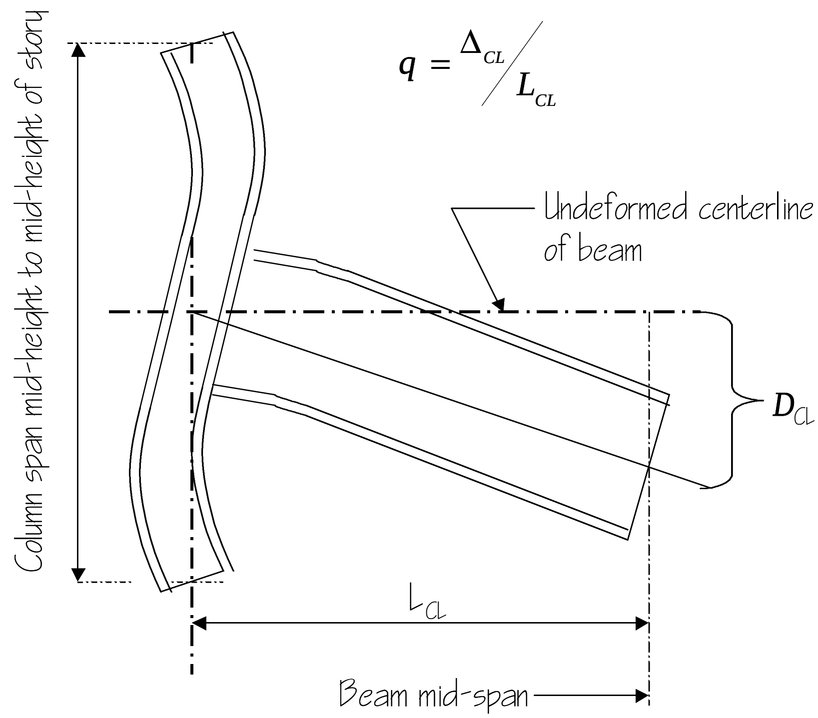


Figure 6-18 Drift Angle

The following modifications and clarifications apply to Appendix S of the 1997 *AISC Seismic Provisions* as modified by Supplement No. 1:

- In lieu of the requirements in Section S5.2, the size of the beam used in the test specimen shall be at least the largest depth and heaviest weight used in the structure. Once the beam is chosen, the test column shall be selected to represent properly the inelastic action anticipated of the column in the real structure, given the chosen beam. Extrapolation beyond the limits stated in this section is not recommended.
- As an alternative to the loading sequence specified in Section S6.3, the FEMA/SAC loading protocol (Krawinkler et al., 2000) is considered acceptable. In the basic loading history, the cycles shall be symmetric in peak deformations. The history is divided into steps and the peak deformation of each step *j* is given as q_j, a predetermined value of the drift angle. The loading history, shown in Table 6-13, is defined by the following parameters:

  q_j = the peak deformation in load step *j*

  n_j = the number of cycles to be performed in load step *j*

Table 6-13 Numerical values of q_j and n_j

Load Step #	Peak deformation q_j	Number of cycles, n_j
1	0.00375	6
2	0.005	6
3	0.0075	6
4	0.01	4
5	0.015	2
6	0.02	2
7	0.03	2

Continue incrementing q in steps of 0.01 radians, and perform two cycles at each step until assembly failure occurs. Failure shall be deemed to occur when the peak loading falls to 20% of that obtained at q_{IO} or if the assembly has degraded to a state at which stability under gravity load becomes uncertain.

Commentary: The AISC Seismic Provisions (AISC, 1997) have been adopted by reference into FEMA-302, 1997 NEHRP Recommended Provisions for New Buildings. The AISC Seismic Provisions include, and require the use of, Appendix S, Qualifying Cyclic Tests of Beam-to-Column and Link-to-Column Connections, for qualification of connections that are not pre-qualified. Appendix S includes a complete commentary on the requirements.

Under Appendix S the test specimen must represent the largest beam anticipated in the project. The column must be selected to provide a flexural strength consistent with the strong-column-weak-beam requirements and panel-zone strength requirements. The permitted weight and size limits contained in Section S5.2 of Appendix S have been eliminated.

AISC loading history and acceptance criteria are described in terms of plastic rotation while the FEMA/SAC loading protocol, acceptance criteria and design recommendations contained in these Recommended Criteria are controlled by total drift angle, as previously defined. The engineer should ensure that the appropriate adjustments are made when using the AISC loading history with these Recommended Criteria. In general, total drift angle is approximately equal to plastic rotation plus 0.01 radians. However, the engineer is cautioned that plastic rotation demand is often measured in different ways and may require transformation to be consistent with the measurements indicated in Figure 6-18.

The calculation of q illustrated in Figure 6-18 assumes that the top and the bottom of the test column are restrained against lateral translation. The height of the test specimen column should be similar to that of the actual story height to *prevent development of unrealistically large contributions to q from flexure of the column.*

6.9.2 Acceptance Criteria

For Simplified Upgrade, the median value of the drift angle capacity at strength degradation, q_{SD}, and at connection failure, q_U, obtained from qualification testing shall not be less than indicated in Table 6-14. The coefficient of variation for these two parameters shall not exceed 10% unless the mean value, less one standard deviation, is also not less than the value indicated in Table 6-14.

Table 6-14 Minimum Qualifying Total Interstory Drift Angle Capacities, q_{SD} and q_U, for OMF and SMF Systems

Structural System	Qualifying Drift Angle Capacity – Strength Degradation, q_{SD} (radians)	Qualifying Drift Angle Capacity – Ultimate, q_U (radians)
OMF	0.02	0.03
SMF	0.04	0.06

Where the clear-span-to-depth ratio of beams in the moment-resisting frame is less than 8, the qualifying total drift angle capacities indicated in Table 6-14 shall be increased to q'_{SD} and q'_U, given by Equations 6-12 and 6-13:

$$q'_{SD} = \frac{8d}{L}\left(1+\frac{L-L'}{L}\right)q_{SD} \qquad (6\text{-}12)$$

$$q'_U = \frac{8d}{L}\left(1+\frac{L-L'}{L}\right)q_U \qquad (6\text{-}13)$$

where: q'_{SD} = Qualifying strength degradation drift angle capacity for spans with $L / d < 8$
q_{SD} = the basic qualifying strength degradation drift angle capacity, in accordance with Table 6-14
q'_U = the qualifying ultimate drift angle capacity, for spans with $L / d < 8$
q_U = the basic qualifying ultimate drift angle capacity, in accordance with Table 6-14
L = the center-to-center spacing of columns, per Figure 6-4, inches.
L' = the distance between points of plastic hinging in the beam, inches.
d = depth of beam in inches

For Systematic Upgrade, the median drift angle capacity for Immediate Occupancy performance level shall be taken as the median value of the drift angle, q_{IO}, at which the peak connection strength occurs, in accordance with Table 6-12. The median drift angle capacity for the Collapse Prevention performance level shall be taken as the median value of the drift angle, q_U, in accordance with Table 6-12. Resistance factors, f, shall be determined in accordance with the procedures of Appendix A of these Recommended Criteria. For any connection, the value of f need not be taken as less than 0.75 for the Immediate Occupancy Level or less than 0.5 for the Collapse Prevention Level.

Commentary: This section sets criteria for use in project-specific qualification of connection and connection upgrade details, in accordance with Section 6.9 and for development of new connection and connection upgrade prequalifications in accordance with Section 6.10 of these Recommended Criteria. Two interstory drift angle capacities are addressed. The values indicated in Table 6-14 formed the basis for extensive probabilistic evaluations of the performance capability of various structural systems, reported in FEMA-355F, State of the Art Report on Performance Prediction and Evaluation. These probabilistic evaluations indicate a high confidence, on the order of 90%, that regular, well-configured frames meeting the requirements of FEMA-302 and constructed with connections having these capabilities, can meet the intended performance objectives with regard to protection against global collapse. They indicate moderate confidence, on the order of 50%, that connections can resist Maximum Considered Earthquake demands without local life-threatening damage.

Connection details with capacities lower than those indicated in this section may be suitable for upgrades to performance criteria other than those that form the basis for FEMA-302. This suitability requires demonstration using the performance evaluation procedures contained in Chapter 3 and Appendix A of these Recommended Criteria.

Connections in frames where beam-span-to-depth ratios are less than those used for the prequalification testing will experience larger flange strains at the plastic hinges, at a particular frame drift, than those tested. For this reason, connections used in such frames need to be qualified for larger drifts as indicated by Equations 6-12 and 6-13, unless the frames are designed to experience proportionally lower drifts than permitted by FEMA-302.

6.9.3 Analytical Prediction of Behavior

Connection qualification should include development of an analytical procedure to predict the limit states of the connection assembly, as demonstrated by the qualification tests. The analytical procedure should permit identification of the strength demands, deformation demands, and limit states on various elements of the assembly at the various stages of behavior. The analytical procedure should be sufficiently detailed to permit design of connections employing members similar to those tested within the limits identified in Section S5.2 of the 1997 *AISC Seismic Provisions*.

Commentary: It is important for the designer to have an understanding of the limiting behavior of any connection detail so that it may be designed and specified on a rational basis for assemblies that vary within specified limits from those tested.

6.10 Prequalification Testing Criteria

This section provides criteria for development of new prequalifications for connection and connection upgrade details for which there is no current prequalification or to extend the parametric limitations for prequalification listed in Section 6.5, for general application. Prequalification includes a program of connection assembly prototype testing supplemented by a suitable analytical procedure that permits prediction of behavior identified in the testing program.

Commentary: The purpose of this section is to provide recommended procedures for prequalification of a connection or connection upgrade detail that is not currently prequalified in these Recommended Criteria or to extend the range of member sizes that may be used with currently pre-qualified connections for general application. These criteria are intended to require significantly more testing than are required for a project-specific qualification program, as once a connection is prequalified, it can have wide application. Prequalification of a connection should incorporate both the testing described in this section and due consideration of the following four criteria:

1. *There should be sufficient experimental and analytical data on the connection's performance to establish the likely yield mechanisms and failure modes for the connection.*

2. *Rational models should be developed and validated for predicting the resistance associated with each mechanism and failure mode.*

3. *Given the material properties and geometry of the connection, a rational procedure should be available to estimate which mode and mechanism controls the behavior and the deformation capacity (i.e., the drift angle) that can be attained from the controlling conditions.*

4. *Given the models and procedures, there should be an adequate data base of experiments to permit assessment of the statistical reliability of the connection.*

The potential for limit states leading to local collapse (i.e., loss of gravity-load capacity) is an important consideration in evaluating the performance of a prototype connection. Establishing this limit state as required by Section 6.9.1 *will necessitate imposing large deformations on the connection. This will require loading setups capable of delivering long strokes while withstanding correspondingly large out-of-plane deformations or large torsional deformations. Many tests are terminated before the ultimate failure of the connection to protect the loading apparatus. These early terminations will limit the range over which a connection may be prequalified.*

6.10.1 Prequalification Testing

Testing and acceptance criteria should follow the recommendations in Section 6.9 except that at least five nonidentical test specimens shall be used. The resulting range of member sizes that will be prequalified should be limited to the range represented by the tested specimens.

6.10.2 Extending the Limits on Prequalified Connections

Once a connection has been prequalified, with its parameters lying within certain ranges, extending this limitation for general use requires further testing. Testing and acceptance criteria should follow the recommendations in Section 6.9 except that at least two nonidentical test specimens shall be tested. The resulting range of member size that will be prequalified should be limited to those contained in the database of tests for the connection type.